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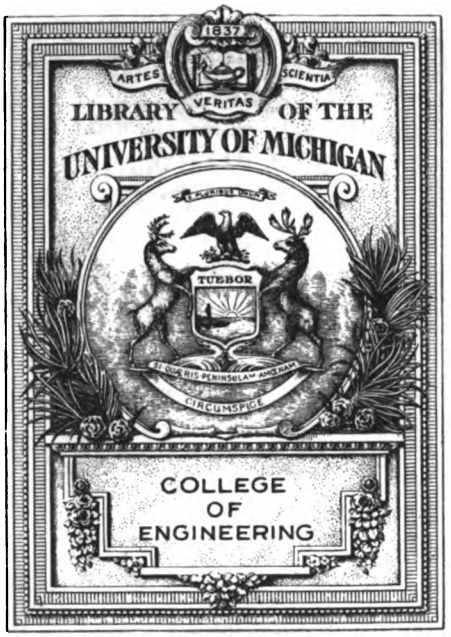

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 171

AUTOMATIC APPARATUS FOR INTERMITTENT TESTING

BY

G. W. VINAL, Associate Physicist
L. M. RITCHIE, Assistant Chemist
Bureau of Standards

JULY 22, 1920

PRICE, 5 CENTS

Sold only by the Superintendent of Documents, Government Printing Office
Washington, D. C.

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FIG. 1.—*Complete view of the new automatic apparatus for intermittent testing. The clock controls the selective relays under the glass case on the table. This apparatus can carry on simultaneously a number of different tests on dry cells and storage batteries. Dry-cell tests are made in the constant-temperature room at the left. The voltage of each group of batteries may be read at the telephone jacks beneath the window*

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AUTOMATIC APPARATUS FOR INTERMITTENT TESTING

By G. W. Vinal and L. M. Ritchie

1. INTRODUCTION AND APPLICATION

This apparatus has been devised to meet the needs of the Bureau of Standards in making tests of dry cells and storage batteries, but is applicable to nearly any form of intermittent test requiring the closing of electrical circuits at regular time intervals.

The usual tests of dry cells which are made at the present time are arbitrary. They involve the discharge of the cells through resistances for varying periods of time, depending upon the service for which the particular cells are designed. The telephone test, frequently referred to as the "A. T. and T. test," consists of a discharge of 3 cells in series during 4 minutes per hour for 10 consecutive hours a day, but omitting every other discharge period on 1 day of the week. The "ignition test" for cells which are designed for a heavier type of service involves a discharge of the group of cells for 1-hour periods twice each day. The "flash-light test" requires a discharge of the cells for 5 minutes once during 24 hours. The details of these tests may be found in Circular No. 79, Electrical Characteristics and Testing of Dry Cells, published by the Bureau of Standards.

Additional tests which may be made simultaneously with these include the life-testing of storage batteries and intermittent tests of caustic-soda primary batteries, such as are used for railway signaling. The life tests of storage batteries consist of a continuous series of charges and discharges, 2 complete cycles being made in each 24 hours, without requiring the presence of an observer. Researches on the capacity of various batteries for differing periods of discharge are planned also.

2. DESCRIPTION OF CONTROL APPARATUS

A general view of the apparatus[1] is shown in Fig. 1. The pendulum clock, in which an electrical contact has been placed, closes the circuit once every minute and furnishes an electrical

[1] We are indebted to Matthew Harrison and F. M. Defandorf for valuable assistance.

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impulse to one of two selective relays which are contained in the glass case on the table. The details of these relays may be more clearly seen in Figs. 2 and 3. As each impulse is received, the shaft at the left-hand end is advanced by one-sixtieth of a revolution, and therefore it makes 1 complete revolution per hour. Once an hour this shaft furnishes a similar impulse to the coils at the other end of the apparatus, which causes the shaft at the right-hand end to rotate. There are 24 teeth in the wheel at this end, and this shaft therefore makes 1 revolution in 24 hours. The third shaft is geared to the 24-hour shaft in the ratio of 1 to 7 and makes 1 revolution in a week. By placing suitably-designed commutators on these three shafts the apparatus may be made to control any required periodic test.

Owing to the inertia of the commutators on these shafts, it was necessary to use a driving mechanism similar to the escapement of a clock, in order that the shaft might not turn by more than the given angular amount when the armature is operated by the current in the magnet coils. The commutators are made of hard rubber, with metallic segments having platinum surfaces. The brushes which bear upon the commutators are of phosphor-bronze, with contact points of platinum-iridium. In the illustration, Fig. 3, the commutators for making the telephone test may be seen. The commutator on the hour shaft closes the circuit for 4 minutes. The commutator on the day shaft has 10 segments which are staggered, every other one of these being made inoperative one day a week by the insulated portion of the commutator on the week shaft. Commutators for the other tests which are being controlled by this apparatus are not shown in this picture.

When the circuit through the contacts on the commutators is complete, the current closes one of the master relays which may be seen in Fig. 1 above the automatic apparatus. These are telephone relays of the type No. 118, Western Electric, to which small phosphor-bronze springs have been added to prevent sticking of the relay owing to the current of the multiple-switch solenoid, which flows through the contact for a considerable time. These springs are operative only at the instant when the relay begins to release the armature and do not affect the relay when the magnetized coil first acts on the armature after the circuit is closed. One master relay is provided for each different test excepting for the storage-battery life-test circuit, for which one relay operates the charging circuit and a second relay the discharge circuit. Each relay is connected with an electric counter (Kellogg

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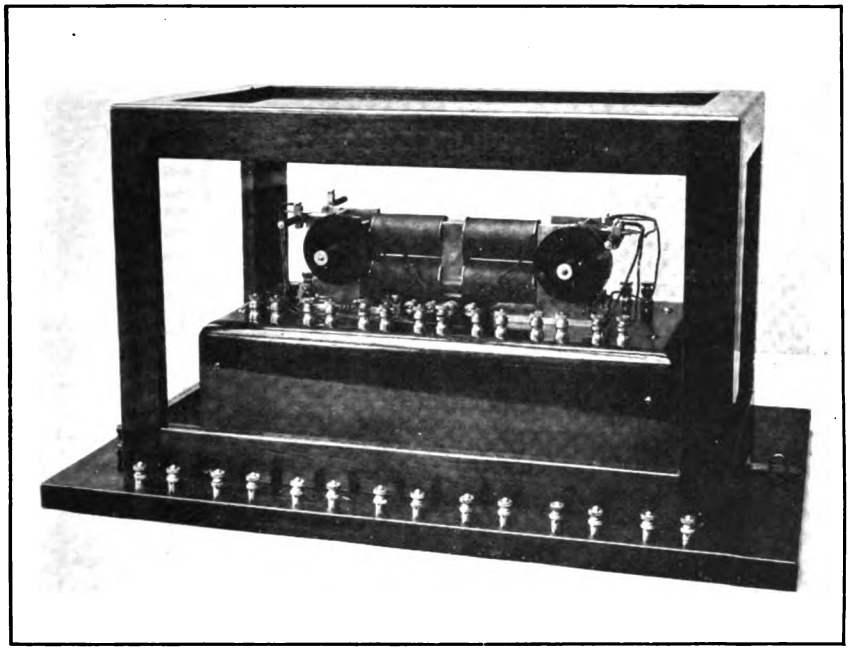

FIG. 2.—*The control apparatus. The shaft at the left makes one revolution per hour, the shaft at the right one revolution per day, and a third shaft at the back (see Fig. 3) one revolution per week*

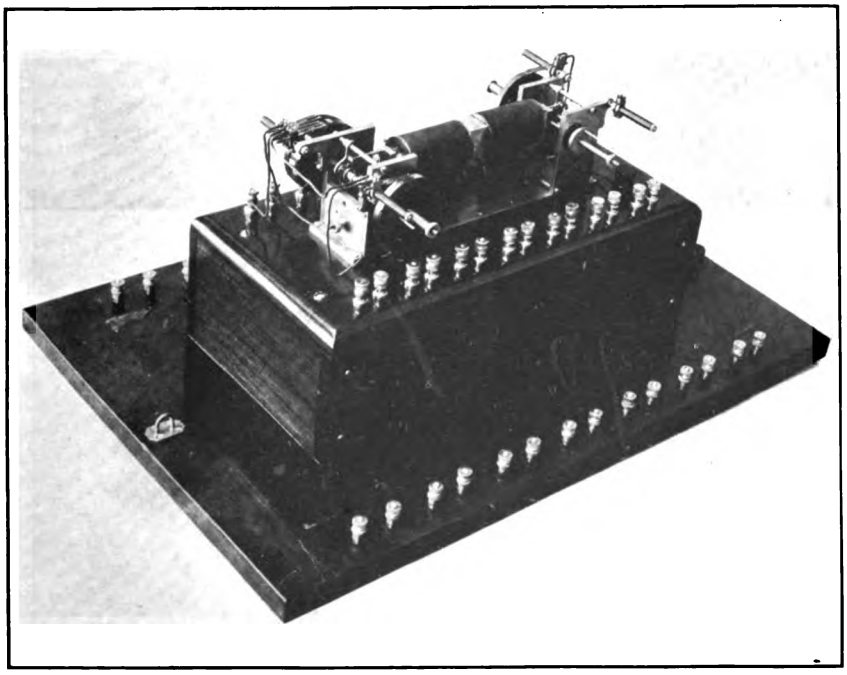

FIG. 3.—*Detail view of the apparatus. The driving mechanism is like the escapement of a clock. Segments on the commutators have platinum surfaces. The brushes are of phosphor-bronze with platinum-iridium contact points. Only a few of the commutators are shown*

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message register), which records the number of times that the circuit is closed. A secondary contact in each message register is utilized to light a small signal lamp to indicate which circuit is in operation.

The diagram of the circuits is shown in Fig. 4. Between the clock and the control apparatus there are two relays. The first is within the clock itself; the second is in the box which may be seen immediately beneath the clock in Fig. 1. This second relay is a matter of convenience for other purposes, but is not necessary for the operation of the apparatus. In the locked box with this

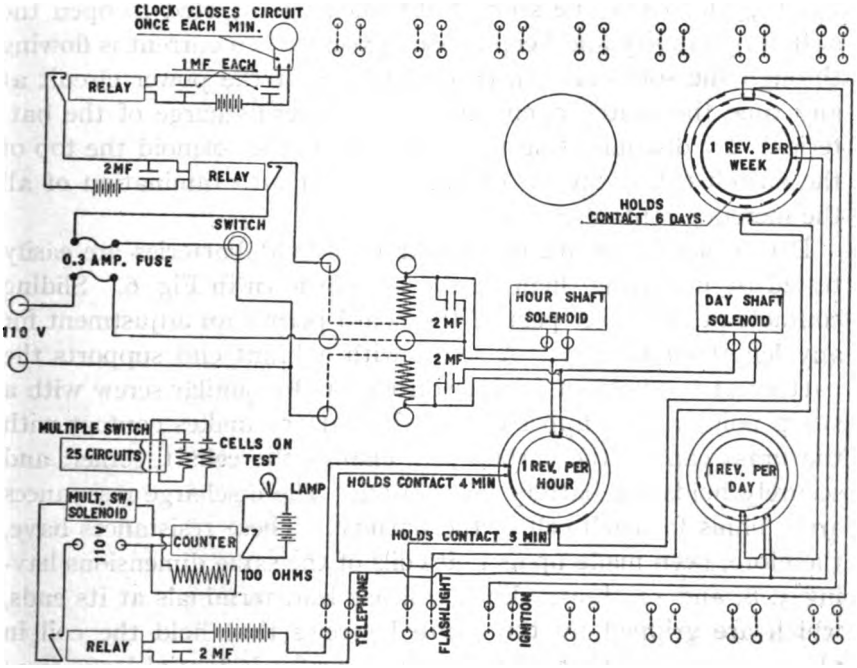

FIG. 4.—*Diagram of the circuits*

relay is a snap switch for the 110-volt circuit, also 2 fuses made of gold foil which will burn out if the current exceeds 0.3 ampere. They are to protect the solenoids of the automatic apparatus.

The batteries indicated in Fig. 4 may be seen beneath the table in Fig. 1. They are of the copper oxide-zinc type, approximately 300 ampere-hour capacity, made by the Edison Primary Battery Co., the National Carbon Co., and the Waterbury Battery Co. They are assembled in a box on casters and have flexible electrical connections so that they may be rolled out into the room for examination or voltage measurements whenever required.

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3. MULTIPLE SWITCHES AND TEST BOARDS

For the dry-cell testing it is necessary to close a large number of independent circuits simultaneously, For this purpose multiple switches, as shown in Fig. 5, are used. The contacts are mercury cups with double break. The switch shown is constructed of "Formica." The upper part of the switch is mounted on pivot hinges and is operated by the solenoid. The current required to operate this solenoid is 0.09 ampere, which is sufficient to give the switch a positive action without being too much current for the secondary contact of the relay. The counterweight, which may be seen at the back, is adjusted to open the switch by gravity and keep it open except when current is flowing through the solenoid. In case of failure of the power circuit at any time the switch opens and avoids overdischarge of the batteries. By disconnecting the switch from the solenoid the top of the switch can easily be thrown back for an examination of all the mercury contacts.[2]

Dry cells such as are in tubular flash-light batteries are easily tested by mounting them on the board shown in Fig. 6. Sliding contacts at the lower part of the board permit an adjustment for any length of battery. A screw with a blunt end supports the battery at the bottom (zinc electrode) and a similar screw with a sharp point at the upper end of the battery makes contact with the brass cap on the carbon rod, clamps the cells together, and securely holds the battery in position. The discharge resistances are 4 ohms for each cell in the battery. These resistances have, therefore, been made up as unit coils of the same dimensions having 4, 8, and 12 ohms. Each coil has lead terminals at its ends, which are gripped by the pointed screws that hold the coil in place—above the battery which is on test. Potential leads from the terminals of each battery pass in a cable along the back of the board and are brought to the telephone jacks beneath the window of the constant-temperature room shown in Fig. 1. Readings are rapidly taken on a voltmeter by the telephone-plug connection while the batteries are discharging.

4. SUMMARY

The apparatus which has been described was designed primarily for testing dry cells and storage batteries. It automatically closes the contacts for the discharge of the dry cells at predetermined

[2] Since this paper was written we have also used 31-contact relays (Type 331H) made by the North Electric Co. These are provided with wiping contacts suitable for closing the battery circuits.

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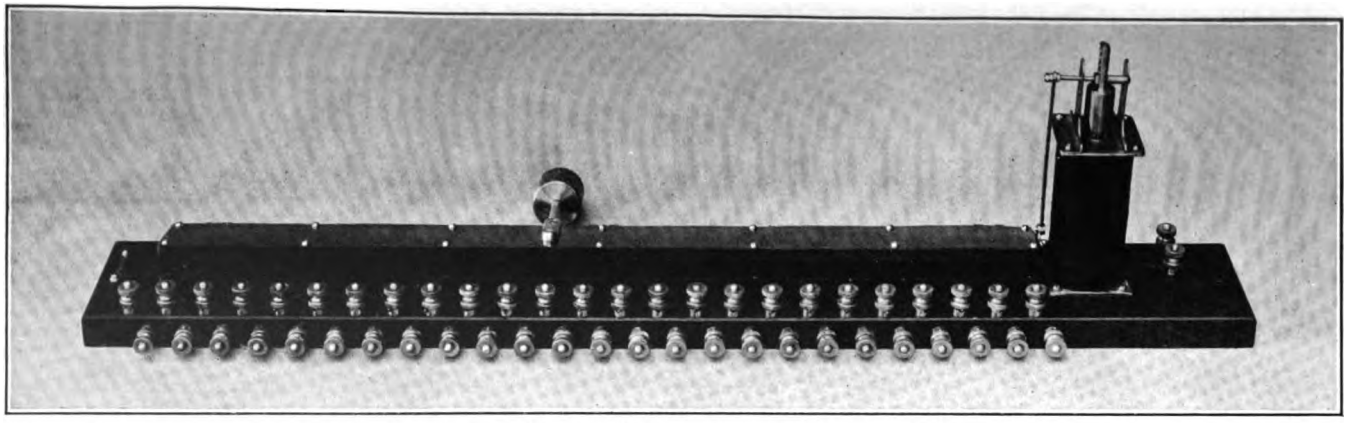

FIG. 5.—*Multiple switch used for dry-cell testing. It is operated by the solenoid at the right, which requires 0.09 ampere. The switch closes 25 circuits for the test board shown in Fig. 6. It is opened by gravity*

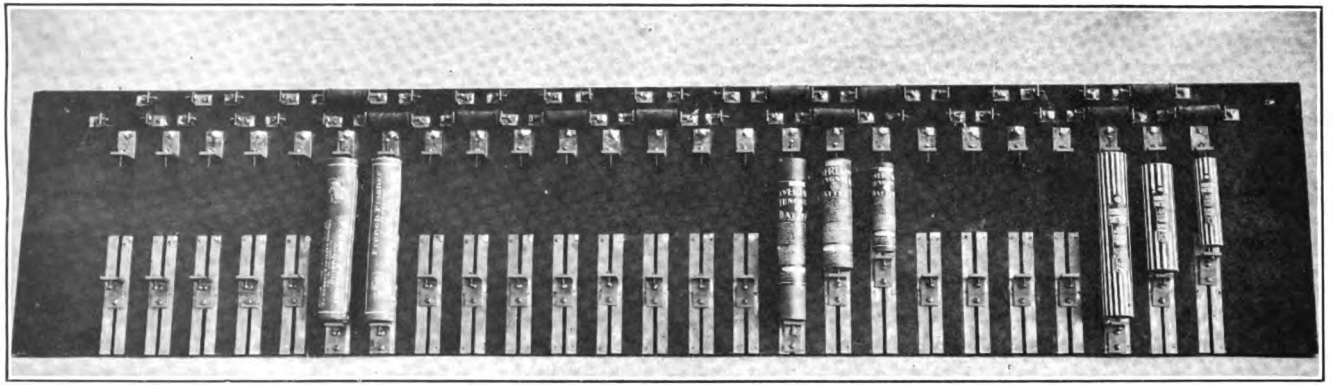

FIG. 6.—*Test board for flashlight batteries. Adjustable contacts provide for any length of battery. Resistance units for the discharge are mounted at the top. Potential leads for each battery are brought to the telephone jacks beneath the window in Fig. 1*

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times and controls both the charge and discharge of storage batteries on life tests. The apparatus consists of 2 selective relays having 3 rotating shafts which make complete revolutions in 1 hour, 1 day, and 1 week, respectively. The coils of the first relay are energized once every minute by the clock shown in Fig. 1. The coils of the second relay are energized once each hour by the first relay. In this way rapidly moving parts are eliminated and the accuracy of the time intervals increased. Any periodic test may be controlled by this apparatus by means of suitably designed commutators which are placed on the rotating shafts. The contacts on the commutators are platinum against platinum iridium. Test boards for various sizes of dry cells with multiple switches to close the individual circuits are described.

WASHINGTON, April 22, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 172

CAST IRON FOR LOCOMOTIVE-CYLINDER PARTS

BY

C. H. STRAND, Associate Physicist
Bureau of Standards

SEPTEMBER 11, 1920

PRICE 10 CENTS

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CAST IRON FOR LOCOMOTIVE-CYLINDER PARTS

By C. H. Strand

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PART 1

INVESTIGATION—METHODS AND RESULTS

I. INTRODUCTION

Frequent renewal of cylinder parts of locomotives results in greatly increased cost of maintenance to the railroads, and consequently the quality of the cast iron entering into their construction is a matter of paramount importance, particularly from the standpoint of wear. These parts include piston-valve bushings, piston-valve packing rings, piston-valve bull rings, cylinder bushings, piston packing rings, and piston-head or bull rings.

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It was found that ordinary high-silicon cast iron gave unsatisfactory wear, particularly in modern superheater locomotives, and the tendency has been toward a harder and stronger iron.

At the request of the former U. S. Railroad Administration the Bureau of Standards has investigated the mechanical, chemical, and microscopical properties of a number of packing rings furnished with service-mileage records, as well as arbitration-test bars, chill-test specimens, and miscellaneous samples from different manufacturers. All of this material was cast iron such as used for the various cylinder parts. It was desired at the same time to review the previous work and specifications on this subject, to ascertain as far as possible the practices of the different foundries and to suggest such revision of existing specifications as would be warranted by the results of the present and of earlier investigations.

The samples, 131 in number, were furnished to the Bureau of Standards by the U. S. Railroad Administration. The first samples were received April 2, 1919, and the last September 26, 1919; they were numbered 1 to 131, inclusive, in order of receipt, and their identification marks and history are to be found in Table 1. In accordance with the policy of the Bureau of Standards, the names of the manufacturers are withheld. In the course of this paper the nine foundries are designated by the letters A to I, inclusive.

TABLE 1.—Identification and History of Material Received for Examination

No.	Manufacturer	Service record	Description
		Miles	
1-26, inclusive	B	93 000	26 pieces constituting a packing ring removed for purpose of this investigation after giving remarkably long service.
27	C	Never in service	A small piece of packing ring of iron claimed to give excellent results.
28	A	....do....	Broken arbitration-test bar representing packing ring and cylinder iron furnished for U. S. Railroad Administration engines. Tests were made in presence of U. S. Railroad Administration inspector, Sept. 4, 1918.
29	A	....do....	Broken arbitration-test bar representing packing ring and cylinder iron furnished for U. S. Railroad Administration engines. Tests were made in presence of U. S. Railroad Administration inspector, Sept. 10, 1918.
30	A	....do....	Broken arbitration-test bar representing packing ring and cylinder iron furnished for U. S. Railroad Administration engines. Tests were made in presence of U. S. Railroad Administration inspector, Oct. 16, 1918.

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TABLE 1—Continued

No.	Manufacturer	Service record	Description
		Miles	
31	A	Never in service	Broken arbitration-test bar representing packing ring and cylinder iron furnished for U. S. Railroad Administration engines. Tests were made in presence of U. S. Railroad Administration inspector, Jan. 17, 1919.
32	A	do	Broken arbitration-test bar representing packing ring and cylinder iron furnished for U. S. Railroad Administration engines. Tests were made in presence of U. S. Railroad Administration inspector, Feb. 4, 1919.
33	A	do	Broken arbitration-test bar representing packing ring and cylinder iron furnished for U. S. Railroad Administration engines. Tests were made in presence of U. S. Railroad Administration inspector, Feb. 15, 1919.
34	A	do	Broken arbitration-test bar representing packing ring and cylinder iron furnished for U. S. Railroad Administration engines. Tests were made in presence of U. S. Railroad Administration inspector, Feb. 20, 1919.
35	A	do	Broken arbitration-test bar representing packing ring and cylinder iron furnished for U. S. Railroad Administration engines. Tests were made in presence of U. S. Railroad Administration inspector, Mar. 10, 1919.
36	A	do	Piece of cylinder packing ring, stamped contract B 1492, for use on U. S. Railroad Administration engines.
37	A	do	Piece of cylinder bushing blank, stamped contract B 1500, for use on U. S. Railroad Administration engines.
38	A	do	Piece of cylinder bushing blank, order 201, for use on U. S. Railroad Administration engines.
39	A	do	Piece of valve-chamber bushing blank, order 201, for use on U. S. Railroad Administration engines.
40	A	do	Piece of packing ring, order 201, for use on U. S. Railroad Administration engines.
41	A	do	Do.
42-43, inclusive	D	17 504	Two entire packing rings in service from May 22, 1918, to Dec. 15, 1918, on a Mikado or 2-8-2 superheated steam locomotive.
44-46, inclusive	C	Never in service	Three new packing rings intended for a superheated engine.
47	C	Apparently in service but no record furnished.	Small piece of packing ring which had apparently been in service.
48-49, inclusive	A	28 988	Two entire packing rings which made "fair" service on engine 5107, Pacific type, superheated.
50-55, inclusive	E	1515	Pieces of packing ring giving poor service on Pacific type engine 5116, superheated.

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TABLE 1—Continued

No.	Manufacturer	Service record	Description
		Miles	
56	B	11 251 20 670 30 793 Average 20 905	Piece of packing ring, stamped 19, from engine using saturated steam.
57	F	5949 4324 6085 Average 5253	Piece of packing ring, stamped No. 39, from engine using superheated steam. Scotch iron.
58	F	10 183 10 798 8824 Average 9935	Piece of packing ring, stamped No. 47, from engine using superheated steam. Common iron.
59	F	22 905 14 938 Average 18 921	Piece of packing ring, stamped No. 16, from engine using saturated steam. Common iron.
60	G	11 012 30 427 Average 20 719	Piece of packing ring, stamped No. 25, from engine using saturated steam. Common iron.
61	G	16 918 21 610 11 251 Average 16 593	Piece of packing ring, stamped No. 27, from engine using saturated steam. Common iron.
62	G	7386 6050 Average 6718	Piece of packing ring, stamped No. 55, from engine using superheated steam.
63	A	15 576 15 943 6832 Average 12 484	Piece of packing ring, stamped No. 33, from engine using superheated steam.
64–69, inclusive	D	Never in service	Six untested arbitration bars of packing-ring iron.
70–75, inclusive	D	do	Six chill-test blocks of packing-ring iron.
76–81, inclusive	C	do	Six untested arbitration bars of packing-ring iron.
82–87, inclusive	A	do	Do.
88–93, inclusive	A	do	Six chill-test blocks of packing-ring iron.
94–99, inclusive	B	do	Six untested arbitration bars of packing-ring iron. Rough castings.
100–103, inclusive	B	do	Four chill-test blocks of packing-ring iron.
104–109, inclusive	H	do	Six untested arbitration bars of packing-ring iron.
110–113, inclusive	H	do	Four chill-test blocks of packing-ring iron.
114–117, inclusive	I	10 000	Four pieces from packing ring from engine 2503.
118–121, inclusive	I	12 000	Four pieces from packing-ring iron, engine 2918, passenger service, using superheated steam.
122–127, inclusive	I	32 000	Six pieces of packing-ring iron, engine 2943, passenger service, using superheated steam.
128–131, inclusive	I	Never in service	Four untested arbitration test bars of packing-ring iron.

II. PREVIOUS WORK

This Bureau thought it desirable to study the previous work upon this subject, and to that end made a survey of the literature. The U. S. Railroad Administration furnished the Bureau with the results of mechanical tests and chemical analyses of a number of melts of air-furnace iron (foundry A) and also with considerable interesting information on the foundry practice of several of the makers whose products were tested in this investigation and on the service-mileage records of packing rings.

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1. REVIEW OF LITERATURE

The most noteworthy contributions to this subject are in the Proceedings of the American Railway Master Mechanics' Association, now the mechanical section of the American Railroad Association. Their original specification, adopted in 1906, required the chemical composition to be within the following limits: Graphitic carbon, 2.75 to 3.25 per cent; combined carbon, 0.50 to 0.70 per cent; silicon, 1.25 to 1.60 per cent; sulphur, 0.06 to 0.10 per cent; phosphorus, 0.50 to 0.80 per cent; manganese, 0.30 to 0.60 per cent. The mechanical requirements of the same specification were a minimum tensile strength of 25 000 pounds per square inch, a minimum transverse breaking load of 3000 pounds, and a minimum deflection of 0.10 inch on the standard 1¼-inch arbitration bar of the American Society for Testing Materials.

The above specifications have undergone several revisions, until their present mechanical and chemical requirements are as indicated in Table 2. In the more recent specifications the composition of the iron is for the most part left to the foundryman, as greater dependence is placed upon the results of the transverse test. A stronger iron is required than in the earlier days.

TABLE 2.—Chemical and Mechanical Characteristics of Cast Iron for Cylinder Parts

[1918 Specifications, American Railway Master Mechanics' Association]

Chemical requirements:		
Phosphorus, maximum	per cent	0.70
Sulphur, maximum	do	.12
Transverse tests, 1¼-inch round bar, 12-inch supports:		
Strength (castings ⅝ inch or less thick), minimum	pounds	3200
Strength (castings over ⅝ inch thick), minimum	do	3500
Deflection (both cases), minimum	inch	.09
Chill in 1¾ by 2 inch block:		
For castings ⅝ inch or less thick, minimum	do	7/32
For castings over ⅝ inch thick	do	⅜

In 1916 a committee of the American Railway Master Mechanics' Association [1] appointed to investigate cast iron for cylinder parts reported in part as follows:

That gun iron (air-furnace iron) is generally considered to be efficient, economical, and for these reasons desirable, can not be doubted, considering the fact that 80 per cent of the roads reporting are using this product in their superheater locomotives for some of the parts mentioned; therefore, the committee does not hesitate to recommend its use for piston-valve bushings, piston-valve packing rings, and pistons or piston bull rings.

[1] Proc. Am. Railway Master Mechanics Assoc., 49, p. 234; 1916.

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Moldenke [2] offers the following explanation of the origin of the term "gun iron":

> This is essentially an air-furnace iron, and, having been put into cast-iron guns from the earliest times, has retained that appellation. At present this grade of metal is used for the most important classes of work, and, being expensive to produce, only the best of pig irons enter into it as a rule.

With regard to cast iron for locomotive cylinders, Moldenke [3] remarks:

> For the latest type of compound engines operated with superheated steam at very high temperatures only the very best of irons should be used, and preferably melted in the air furnace. The metal must be strong and sound, as otherwise leaks develop, which cause loss of power to a serious extent. Moreover, the iron is under very high temperatures and, unless low in graphite, will give serious trouble from "growing."

Hurst [4] has published an interesting paper, which, however, deals mostly with cast iron for cylinders and pistons for internal combustion engines, where the temperatures are considerably higher than in superheated steam locomotive cylinders.

2. DATA FURNISHED BY U. S. RAILROAD ADMINISTRATION

(*a*) FOUNDRY PRACTICE.—The U. S. Railroad Administration advised this Bureau that of the nine foundries supplying material for this investigation, A is the only maker which used the air furnace; all others use the cupola process, as far as could be ascertained.

The cupola is a vertical-shaft furnace lined with refractory material; an opening is provided in the side of the furnace near the top for charging fuel and iron. An air blast is introduced near the bottom, where the molten metal is tapped. In this process of manufacture the iron is in intimate contact with the fuel and absorbs from it such objectionable impurities as sulphur. The air furnace, on the other hand, is of the reverberatory type, and the fire is at one end and the chimney at the other end of the hearth. The flame sweeps across the furnace and furnishes the heat necessary to melt the metal. It is obvious that the iron and fuel are not in as close contact as in the cupola. Other methods of melting in use by the gray-iron foundry are the crucible, the open-hearth, and the electric furnace. As no very considerable percentage of gray-iron castings are made by any of these processes, they are not described in this paper. The electric furnace, however, is rapidly coming to the foreground in the iron foundry.

---

[2] Richard Moldenke, Principles of Iron Founding, p. 191; 1917. McGraw Hill Book Co.

[3] Principles of Iron Founding, p. 193; 1917.

[4] J. E. Hurst, Growth of Cast Iron by Repeated Heating, Proc. Staffordshire Iron and Steel Inst., **32**, pp. 26–54; 1918. Also, Foundry, **46**, pp. 227–229; 1918.

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Manufacturer A, who, as previously stated, uses the air furnace, claims that no steel or ferroalloys are added for the purpose of furnishing strength and that no difficulty is experienced in obtaining a transverse strength of arbitration bar of 4000 pounds, or a tensile strength of 32 000 pounds per square inch; also that the sulphur content seldom exceeds 0.06 per cent. Foundry C states that his practice is to add 2 per cent of an 80 per cent ferrosilicon to approximately 4000 pounds of metal, also claiming that great care should be exercised in skimming and gating when pouring packing rings and bushings. Manufacturer D, who states that he uses 15 per cent of steel scrap in the cupola, furnishes the following additional information:

Cylinder and valve rings are made in the form of pot castings in green-sand molds. Piston-valve bushings are also cast in green-sand molds, and the steam ports are cored with the admission and the exhaust ports both on one core, and the mold is poured at the center. The cylinder bushings are cast on end with dry-sand molds, and the pouring of metal is done as rapidly as possible; the bushings are cast without steam ports.

(*b*) TESTS MADE BY U. S. RAILROAD ADMINISTRATION.—One of the railroad inspectors tested samples from 23 casts of air-furnace iron made between July 15, 1918, and March 10, 1919, with the results shown in Table 3.

TABLE 3.—Results of Air-Furnace Iron Tests by U. S. Railroad Administration

Items determined	Average	Maximum	Minimum
Transverse strength of 1¼-inch arbitration bar in pounds, using 12-inch supports	4300	4939	3583
Transverse modulus of rupture of arbitration bar, pounds per square inch	67 300	77 200	56 000
Deflection, inch	.117	.144	.089
Tensile strength, pounds per square inch	33 400	38 000	27 000
Total carbon, per cent	3.23	3.50	2.84
Graphitic carbon, per cent	2.39	2.73	1.94
Combined carbon, per cent	.84	1.02	.89
Silicon, per cent	1.26	1.57	.95
Sulphur, per cent	.052	.100	.036
Phosphorus, per cent	.39	.49	.30
Manganese, per cent	.40	.52	26

In these heats the strength of arbitration bar was below 4000 pounds in only two instances (3583 and 3720 pounds), and the sulphur content only twice exceeded 0.06 per cent (0.062 and 0.100 per cent).

Several of the railroads made service tests of packing rings, and these records, together with the samples, were kindly furnished

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to the Bureau through the U. S. Railroad Administration. At this point it would, perhaps, be desirable to emphasize the fact that many other factors besides the quality of the iron have considerable bearing upon the results of these tests; such factors are design, lubrication, method of handling locomotive, level or hilly country, good and bad water districts, etc. As a consequence, these facts are to be borne in mind in endeavoring to establish a correlation between laboratory and service tests.

III. INVESTIGATION AT BUREAU OF STANDARDS

1. MECHANICAL TESTS

The mechanical tests made at the Bureau of Standards consisted of transverse, tension, hardness, and fracture of chill-test specimens furnished by the U. S. Railroad Administration.

(*a*) TRANSVERSE TESTS.—*Arbitration Bars.*—Thirty-four arbitration bars, ordered made in accordance with the American Society for Testing Materials standards, from six different manufacturers, were tested (Tables 4 and 5). The foundries were A, H, B, C, I, and D, the preceding order indicating the quality, the first-named being the highest in transverse strength in terms of the modulus of rupture in pounds per square inch. The bars from H showed a higher transverse breaking load in pounds than those from A, but, when the variation in diameter of bar is taken into consideration, the latter is found to be the stronger material. The bars from foundries C and D were unfortunately machined before testing, and, as a consequence, the average strength of the bar is decreased because of the absence of skin hardness. The iron from both sources, however, was of low strength, which fact was amply demonstrated by subsequent tests. The iron from foundry I was irregular in quality, while that from D and H was spongy.

TABLE 4.—Transverse Tests of Unmachined Arbitration Bars

[Span 12-inch, centrally applied load]

No.	Manufacturer	Diameter	Breaking load	Modulus of rupture	Maximum deflection
		Inches	Pounds	Lbs./in.²	Inch
82	A	1.23	3836	63 000	0.117
83	A	1.22	3900	65 600	.110
84	A	1.23	4080	67 000	.122
85	A	1.22	4220	71 000	.136
86	A	1.22	4414	74 200	.132
87	A	1.23	4116	68 000	.132
Average		1.23	4094	68 100	.125

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TABLE 4—Continued

No.	Manufacturer	Diameter	Breaking load	Modulus of rupture	Maximum deflection
		Inches	Pounds	Lbs./in.²	Inch
94	B	1.25	3600	56 300	0.101
95	B	1.27	4280	63 900	.138
96	B	1.25	4210	65 800	.137
97	B	1.25	3986	62 300	.119
98	B	1.27	4160	62 000	.124
99	B	1.25	3480	54 400	.095
Average		1.26	3953	60 800	.119
104	H	1.28	4320	63 000	.099
105	H	1.29	4320	61 500	.104
106	H	1.27	4350	63 600	.111
107	H	1.26	4250	64 900	.118
108	H	1.27	4360	65 100	.115
109	H	1.31	4430	60 200	.102
Average		1.28	4323	63 100	.108
128	I	1.25	4140	64 800	.125
129	I	1.27	2500	37 300	.075
130	I	1.26	3600	55 000	.104
131	I	1.27	3160	47 100	.097
Average		1.26	3325	51 100	.100

TABLE 5.—Transverse Tests of Machined Arbitration Bars

[Span 12-inch, centrally applied load]

No.	Manufacturer	Cross section		Breaking load	Modulus of rupture
		Width	Height		
		Inch	Inch	Pounds	Lbs./in.²
64	D	0.865	0.860	1604	45 100
65	D	.875	.873	1714	46 200
66	D	.856	.865	1580	44 400
67	D	.857	.858	1590	44 700
68	D	.864	.860	1794	50 800
69	D	.854	.859	1630	46 600
Average		.862	.862	1652	46 300
76	C	.854	.851	1850	53 800
77	C	.852	.853	1565	45 500
78	C	.854	.854	1864	53 800
79	C	.853	.852	1820	52 900
80	C	.852	.852	1830	53 200
81	C	.863	.850	1650	47 600
Average		.855	.852	1763	51 100

Packing Rings.—When the packing rings were sufficiently large in size, specimens for small transverse tests were machined from them, as indicated in Fig. 1. The reader is referred to Table 6 for the results of the tests, together with the service records.

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It is not to be expected that these results will be similar to those on unmachined arbitration bars, because of the absence of skin or surface hardness. The rings from manufacturer A, which gave fairly good service, had a transverse strength of 48 900 pounds per square inch; B, 47 400 pounds per square inch; C, 46 800

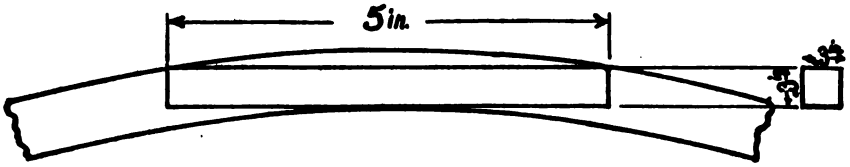

FIG. 1.—*Method of cutting small transverse test pieces from packing ring*

pounds per square inch; and D, 40 600 pounds per square inch. It is apparent from these tests that the transverse strength of the ring from foundry B, which gave the excellent service record of 93 000 miles, was not exceptional.

TABLE 6.—Transverse Tests of Packing Rings

[Span 4 inches, centrally applied load]

No.	Manufacturer	Cross section		Modulus of rupture	Maximum deflection	Service record
		Width	Height			
		Inch	Inch	Lbs./in.²	Inch	Miles
42	D	0.311	0.309	41 200	0.070	17 504
43	D	.308	.309	40 000	.073	17 504
Average				40 600		
44	C	.304	.307	45 300	.085	
44	C	.313	.307	44 400	.073	
45	C	.304	.305	53 500	.072	
45	C	.306	.306	44 600	.071	
46	C	.313	.312	45 300	.074	
46	C	.314	.313	42 900	.071	
47	C	.313	.308	51 800	.068	
Average				46 800		
48	A	.297	.312	50 000	.092	28 988
48	A	.312	.306	50 300	.074	28 988
49	A	.313	.308	46 500	.077	28 988
Average				48 900		
2	B	.298	.302	46 200	.071	93 000
24	B	.287	.292	47 100	.073	93 000
56	B	.276	.312	48 900	.070	20 905
Average				47 400		

(*b*) TENSION TESTS.—*Arbitration Bars.*—After breaking the arbitration bars in the transverse tests the broken pieces were used to determine the tensile strength of the material. The results of the tests of the products of six of the foundries are summarized

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in Table 7. The iron from A, as in practically all of the tests, showed some slight superiority, with an average tensile strength of 34 600 pounds per square inch. The others in order of decreasing strength were B, 33 500 pounds per square inch; I, 32 050 pounds per square inch; H, 30 700 pounds per square inch; D, 25 750 pounds per square inch; and C, 25 050 pounds per square inch.

TABLE 7.—Tension Tests of Arbitration Bars

No.	Manufacturer	Diameter	Tensile strength	No.	Manufacturer	Diameter	Tensile strength
		Inch	Lbs./in.²			Inch	Lbs./in.²
82	A	0.505	33 300	94	B	0.505	32 900
84	A	.503	35 900	97	B	.502	34 100
Average		.505	34 600	Average		.503	33 500
64	D	.501	24 800	106	H	.503	30 600
69	D	.505	27 200	108	H	.503	30 800
Average		.503	25 750	Average		.503	30 700
77	C	.505	26 200	128	I	.503	34 600
79	C	.505	23 900	129	I	.505	30 600
Average		.505	25 050	130	I	.505	32 100
				131	I	.503	30 900
				Average		.504	32 050

Packing Rings.—Tension tests were made from packing rings furnished by foundries A, B, F, and G, and the results, together with service records, are to be found in Table 8. The ring from foundry B, which gave excellent service, also had the highest tensile strength; namely, 25 450 pounds per square inch. The other rings, all of which gave poor results in the service tests, were likewise inferior in tensile strength. From these tests, at least, there appears to be some more or less definite relation between laboratory and service tests.

TABLE 8.—Tension Tests of Packing Rings

No.	Manufacturer	Diameter	Tensile strength	Service record
		Inch	Lbs./in.²	Miles
5	B	0.293	27 500	93 000
7	B	.300	23 400	93 000
13	B	.299	25 500	93 000
Average			25 450	
57	F	.505	18 500	5253
58	F	.505	18 500	9935
Average			18 500	
62	G	.505	21 900	6718
63	A	.501	16 700	12 484

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(*c*) BRINELL HARDNESS TESTS.—*Arbitration bars.*—Brinell hardness tests, using a 10 mm ball with a pressure of 3000 kg, sustained for 30 seconds, were made on flat, longitudinal sections of the arbitration bars. Approximately one-fourth of an inch of material was removed by the shaper to obtain a surface for the Brinell tests. In Table 9 it will be seen that the iron from A was hardest, with a Brinell hardness numeral of 213, H and I next, each with a hardness of 211, and that C was softest, with a Brinell hardness numeral of 203. As there is a variation of only 10 points from the maximum to minimum hardness, it is concluded that, in view of the widely varying quality of the irons as indicated by other tests, the Brinell test does not satisfactorily measure the quality of iron for cylinder parts.

TABLE 9.—Brinell Hardness of Arbitration Bars

[3000 kg pressure for 30 seconds, 10 mm ball]

No.	Manufacturer	Brinell hardness numerals [a]	No.	Manufacturer	Brinell hardness numerals [a]
64	D	206	94	B	207
65	D	192	95	B	211
66	D	207	96	B	207
67	D	204	97	B	206
68	D	207	98	B	226
69	D	216	99	B	197
Average		205	Average		209
76	C	206	104	H	214
77	C	192	105	H	212
78	C	206	106	H	213
79	C	206	108	H	205
80	C	205	108	H	211
81	C	204	109	H	212
Average		203	Average		211
82	A	204	128	I	210
83	A	216	129	I	207
84	A	213	130	I	219
85	A	212	131	I	210
86	A	209	Average		212
87	A	226			
Average		213			

[a] Average of 3 determinations.

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TABLE 10.—Brinell Hardness of Packing Rings, Bushings, Etc.

[3000 kg pressure for 30 seconds, 10 mm ball]

No.	Manufacturer	Brinell hardness numeral	Service record	No.	Manufacturer	Brinell hardness numeral	Service record
			Miles				Miles
1	B	175	93 000	42	D	133	17 504
5	B	163	93 000	43	D	160	17 504
7	B	165	93 000				
13	B	166	93 000	Average		147	
56	B	167	20 905				
				51	E	146	1516
Average		167					
				57	F	141	5253
				58	F	159	9935
27	C	186		59	F	166	18 921
44	C	159					
45	C	160		Average		155	
46	C	163					
47	C	187		60	G	171	20 719
				61	G	183	16 593
Average		180		62	G	177	6718
36	A	175		Average		177	
37	A	153					
38	A	155		114–117, inclusive	I	172	10 000
39	A	180		118–121, inclusive	I	158	12 000
40	A	170		122–127, inclusive	I	174	32 000
41	A	179					
48	A	161	28 988	Average		168	
49	A	170	28 988				
Average		168					

Packing Rings, Bushings, and Similar Parts.—The Brinell hardness tests of packing rings, bushings, etc. (Table 10), indicate that the castings themselves are considerably softer than the arbitration bars. This is undoubtedly due to the fact that the slower rate of cooling of the former results in a higher graphitic carbon content. This is particularly true of some of the samples from foundry A, which were cylinder bushings, having a considerably larger cross-sectional area than packing rings.

(*d*) CHILL TESTS.—Chill-test specimens were received from four foundries. Foundries A and H used the standard chill-test specimen of the American Society for Testing Materials, while B and D used specimens not standard in size. Details are shown in the drawings in Fig. 2. The specimens were broken at the Bureau of Standards, four to six samples of the product of each foundry having been submitted. The iron from makers A and D showed an average depth of chill of one-half an inch, and that from B and H had a chill of approximately one-fourth of an inch.

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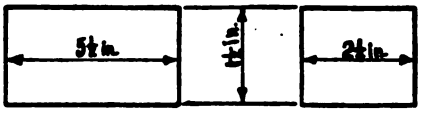
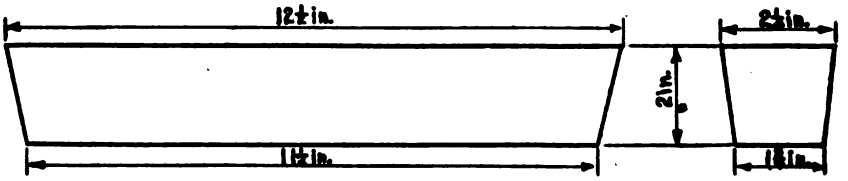
2. CHEMICAL COMPOSITION

(*a*) ARBITRATION BARS.—Chemical analyses were made of one of each of the arbitration bars (Table 11) from each of six foundries, except in the case of I, where two bars were analyzed because of

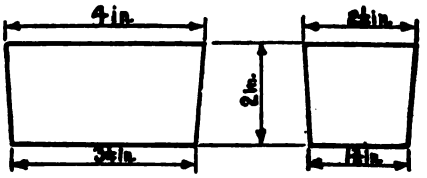


Manufacturer "D" specimens 70-75 inclusive



Manufacturer "A" specimens 88-93 inclusive

Manufacturer "H" specimens 110-113 inclusive



Manufacturer "B" specimens 100-103 inclusive

FIG. 2.—*Dimensions of three types of chill-test specimens from various manufacturers*

the widely varying mechanical properties, as shown in Table 4. The poor quality of the iron in specimen No. 129 (foundry I) is attributable to its high silicon content.

TABLE 11.—Chemical Composition of Arbitration Bars

No.	Manufacturer	Carbon			Silicon	Sulphur	Phosphorus	Manganese
		Total	Graph.	Combined				
		Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
64	D	3.36	2.55	0.81	1.42	0.093	0.48	0.61
77	C	3.36	2.40	.96	1.33	.167	.56	.39
84	A	3.30	2.39	.91	1.13	.051	.51	.36
94	B	3.30	2.50	.80	1.36	.124	.37	.90
106	H	3.32	2.42	.90	1.03	.137	.52	.54
128	I	3.26	2.37	.89	1.00	.161	.38	.63
129	I	3.31	2.66	.65	1.47	.132	.50	.53

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The various bars were quite uniform in total, graphitic, and combined carbon content. The silicon and sulphur contents, however, varied within rather wide limits, the former from 1 per cent to 1.47 per cent (both samples from I) and the latter from 0.051 per cent (A) to 0.167 per cent (C). The iron from manufacturer B was the lowest in phosphorus content, with 0.37 per cent, and that from C was highest, with 0.56 per cent. Manganese varied from 0.36 per cent in the product from A to 0.90 per cent in the material from foundry B.

It is generally recognized that there should be at least three times the percentage of manganese as sulphur present; otherwise, sulphur will combine with iron instead of manganese, and brittle material will result. It will be observed that in the iron from C there is approximately only twice as much manganese as sulphur present. This condition, together with the high combined carbon content, accounts, in all probability, for the poor showing of this iron in the mechanical tests (Tables 5 and 7).

(*b*) PACKING RINGS, BUSHINGS, AND SIMILAR PARTS.—In general (Table 12), the silicon, and naturally the graphitic carbon, is higher in the packing rings, bushings, and similar castings than in the arbitration bars. The ring from B, which gave 93 000 miles in service, showed no unusual characteristics in chemical composition. The rings from foundry C were high in phosphorus and sulphur, as in the case of the arbitration bars. The castings from B were low in sulphur, and the ring which had a service record of 28 988 miles was rather low in combined carbon content. D's rings were high in sulphur and graphitic carbon. The latter condition (high graphitic carbon), of course makes too soft a ring for prolonged wear. The locomotive packing ring from foundry E, which gave a service of only 1516 miles, was very high in silicon (2.84 per cent), and consequently low in combined carbon (0.33 per cent). The rings from I (32 000 miles of service) were fairly low in graphitic carbon content.

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TABLE 12.—Chemical Composition of Packing Rings, Bushings, Etc.

No.	Manufacturer	Carbon			Silicon	Sulphur	Phosphorus	Manganese	Service record
		Total	Graph.	Combined					
		Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Miles
1	B	3.46	2.79	0.67	1.68	0.120	0.260	0.56	93 000
5	B	3.39	2.80	.59	1.67	.140	.260	.57	93 000
7	B	3.53	2.91	.62	1.67	.150	.250	.57	93 000
13	B	3.43	2.69	.74	1.69	.130	.250	.58	93 000
56	B	3.08	2.67	.41	1.70	.124	.825	.43	20 905
27	C	3.36	2.42	.94	1.47	.190	.310	.42	.....
44	C	3.57	2.88	.69	1.51	.130	.601	.51	.....
45	C	3.45	2.84	.61	1.95	.130	.582	.51	.....
47	C	3.44	2.87	.57	1.46	.136	.639	.56	.....
29	A	3.33	2.42	.91	1.21	.060	.250	.38	.....
36	A	3.24	2.40	.84	1.06	.071	.220	.24	.....
37	A	3.37	2.75	.62	1.24	.068	.260	.28	.....
38	A	3.31	2.90	.41	1.50	.068	.230	.54	.....
39	A	3.24	2.65	.59	1.21	.150	.220	.45	.....
40	A	3.31	2.56	.75	.85	.055	.220	.40	.....
41	A	3.28	2.55	.73	.87	.062	.210	.61	.....
48	A	3.46	2.91	.55	1.52	.062	.502	.45	28 988
63	A	3.51	2.84	.67	1.40	.102	.440	.38	12 484
42	D	3.47	3.12	.35	1.83	.151	.668	.56	17 504
43	D	3.50	3.10	.40	1.94	.165	.704	.55	17 504
50	E	3.34	3.01	.33	2.84	.068	.675	.55	1516
57	F	3.40	2.74	.66	1.58	.117	.410	.35	5253
58	F	3.24	2.73	.51	2.14	.119	.855	.32	9935
62	G	3.47	2.68	.79	1.33	.106	.380	.42	6718
116	I	3.22	2.53	.69	1.83	.112	.670	.34	10 000
117	I	3.11	2.54	.57	1.78	.116	.670	.34	10 000
118	I	3.16	2.54	.62	1.22	.140	.360	.52	13 000
121	I	3.29	2.53	.76	1.25	.140	.370	.52	12 000
122	I	3.35	2.57	.78	1.96	.087	.740	.44	32 000
125	I	3.38	2.76	.62	1.95	.091	.760	.43	32 000

3. METALLOGRAPHIC EXAMINATION

Specimens from packing rings made by B (No. 5, 93 000 miles), A (No. 48, 28 988 miles; No. 63, 12 484 miles), D (No. 42, 17 504 miles), G (No. 62, 6718 miles), and E (No. 50, 1516 miles) were examined microscopically.

The samples were polished and examined microscopically both before and after etching. Heat tinting was found to be very suitable for revealing the irregularly shaped masses of phosphide eutectic and at the same time allowing the black flakes of graphite to stand out clearly. Micrographs were taken only of typical

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FIG. 3.—*Specimen No. 5*

Manufacturer "B" (93 000 miles service). Etched with 2 per cent nitric acid. ×100

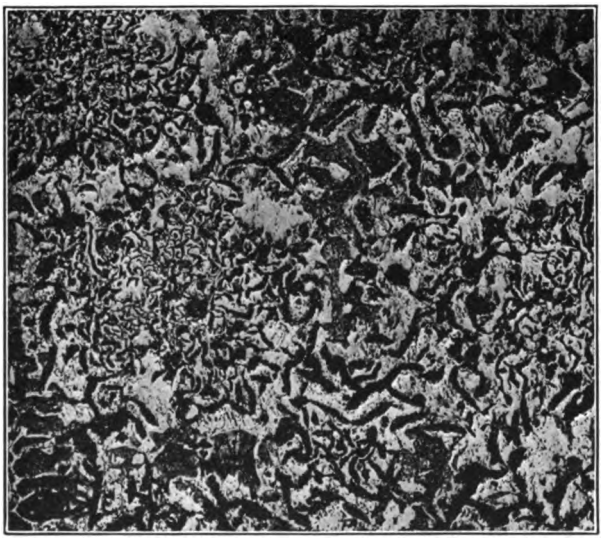

FIG. 4.—*Specimen No. 50*

Manufacturer "E" (1516 miles service). Etched with 2 per cent nitric acid. ×100

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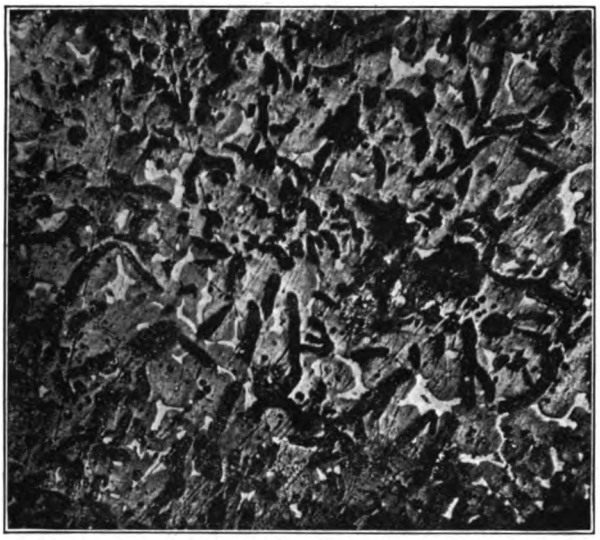

FIG. 5.—*Specimen No. 42*

Manufacturer "D" (17 504 miles service). Heat tinted. ×50

FIG. 6.—*Specimen No. 42*

Manufacturer "D" (17 504 miles service). Etched with 2 per cent nitric acid. ×100

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samples selected for purposes of illustration. Fifty diameters magnification was used to show the size, distribution, and amount of graphite flakes, and 100 diameters to show the constitution of the matrix.

Some attempt was made to correlate the microstructure with the endurance in service, but, due to the many variable factors, it was realized that any very close comparisons were apt to be misleading. The inhomogeneity of cast iron made it somewhat doubtful that the small sample used for metallographic examination was representative of the whole. In some instances, however, with the aid of the principles governing the constitution of gray cast iron, the samples could be approximately classified, and the service records bore out fairly well the predictions that could be made of mechanical qualities from a study of the microstructure.

In Figs. 3 and 4 are shown samples Nos. 5 and 50 after etching with 2 per cent nitric acid. Fig. 3 is the microstructure of the ring which gave 93 000 miles of service, while Fig. 4 shows the ring which gave only 1516 miles' service. The difference between them is striking. In Fig. 3 the gray areas are pearlite, the pure white is ferrite, the irregular speckled masses are phosphide, and the large, black flakes are graphite. In Fig. 4 the white constituent, rather large in amount, is ferrite, the gray areas forming the easily visible network are pearlite, the irregular lighter-colored masses are phosphide, and the small, black flakes are graphite. As might be inferred from the chemical composition (No. 5, 0.59 per cent combined carbon; No. 50, 0.33 per cent combined carbon) the matrix of the former is largely pearlitic, with very little ferrite, while the reverse is true in the latter. The rate of cooling in the latter sample (No. 50) was the more rapid, and thus the finely divided condition of the graphite is to be explained. The very high silicon content of 2.48 per cent accounts for the high graphitic carbon content. This resulted in a soft iron, which, together with the physical unsoundness of the material as revealed by microscopical examination, accounted for the poor service record of 1516 miles. B's ring, on the other hand, while higher in combined carbon (pearlite) and lower in graphite or free carbon, under the microscope shows no explanation for the extraordinarily long service obtained.

The microstructure of the ring from D (No. 42, 17 504 miles) is shown in Figs. 5 and 6. While the combined carbon content is low the distribution of the various constituents is much more uniform than in Fig. 4. The ring, however, was too soft to give

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good wearing qualities. In Fig. 7 is shown the structure of A's ring, sample No. 48, which gave 28 988 miles of service. It does not materially differ in microstructure from the other rings giving fair service; it is, in fact, as indicated by Brinell hardness test and chemical composition, somewhat too low in combined carbon for maximum wear. Fig. 8 shows the microstructure of a ring made by G (6718 miles). The distribution of the graphite is quite uniform, and from a metallographic point of view it is impossible to account for the poor results reported in service.

The microscope is most useful in determining the size, amount, and distribution of graphite flakes, amount of combined carbon, and to some extent the physical soundness of the castings.

IV. COMPARISON OF PROPERTIES OF AIR-FURNACE AND CUPOLA CAST IRON

From a study of the data given in the various tables of tests, it will be observed that the air-furnace iron is somewhat more uniform in character and of generally better quality than cupola iron. A comparison of the two kinds of iron, bearing out this conclusion, and which was computed from all tests of arbitration bars mentioned in this paper, is given in Table 13.

TABLE 13.—Properties of Air-Furnace Iron and of Cupola Cast Iron

Process of manufacture	Transverse test, modulus of rupture			Tensile strength		
	Average	Maximum	Minimum	Average	Maximum	Minimum
	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²
Air furnace	67 500	77 200	56 000	33 500	37 500	27 000
Cupola	54 700	65 800	37 300	29 700	34 600	23 900

V. CONCLUSIONS AND RECOMMENDATIONS

Conclusions are drawn and recommendations made as follows:

1. On the basis of tests made by inspectors of the U. S. Railroad Administration and this Bureau, which were substantially in agreement, it is concluded that air furnace or so-called "gun iron" is more uniform in character and on the average of somewhat better mechanical properties than cupola iron. The latter, however, often equals or even excels in mechanical properties the specimens of air-furnace iron tested in this investigation. The sulphur content of the air-furnace irons examined seldom exceeded 0.06 per cent, while the cupola irons varied in sulphur content from 0.10 to 0.17 per cent.

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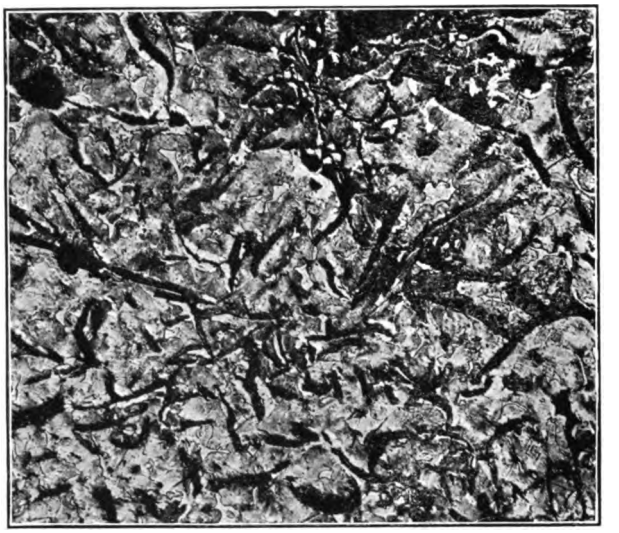

FIG. 7.—*Specimen No. 48*

Manufacturer "A" (28 988 miles service). Heat tinted. ×50

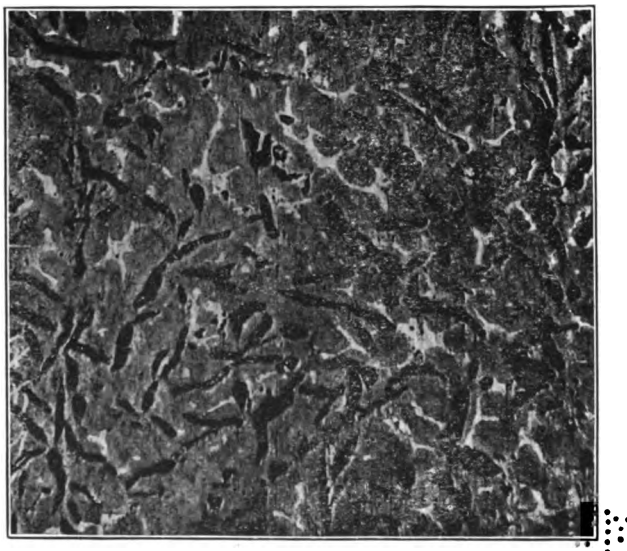

FIG. 8.—*Specimen No. 62*

Manufacturer "G" (6718 miles service). Heat tinted. ×50

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2. The findings of the American Railway Master Mechanics' Association with respect to correlation of laboratory and service tests are confirmed in the present investigation. It was impossible, except in a very general way to find any correlation between the quality of the iron as developed by laboratory tests and the mileage obtained in service. This is explained by the fact that many other factors besides the quality of the iron enter into consideration in the service results; namely, design, lubrication, method of handling the locomotive, topography of the country, character of water used in locomotive, etc. The ring from foundry B, which gave the exceptionally good service of 93 000 miles, showed no unusual properties in laboratory tests. It is for the reasons cited above that the conclusions and recommendations of this paper are based essentially upon the results of laboratory tests.

3. The present specifications of the American Railway Master Mechanics' Association are somewhat lax in the requirements for mechanical properties. It is recommended that the transverse-strength requirements of 1¼-inch arbitration bar be increased from 3200 to 3500 pounds for castings one-half of an inch or less in thickness, and from 3500 to 3800 pounds for castings over one-half of an inch in thickness. It is further recommended that the minimum deflection requirements for both cases be increased from 0.09 to 0.11 inch. The division line of the casting thickness is changed from five-eighths of an inch of the American Railway Master Mechanics' Association, to one-half of an inch in order to conform to the recognized standards of the American Society for Testing Materials.

4. It is preferable to leave the chemical composition and the melting process used to the manufacturer, depending for the most part on the mechanical tests, and of these primarily upon the transverse test. The existing specifications allow a maximum of 0.70 per cent phosphorus and 0.12 per cent sulphur; there are no developments in this investigation which would warrant a revision of the maximum permissible amounts of these elements.

Acknowledgment is due to Capt. S. N. Petrenko and T. W. Greene for the mechanical tests and also to S. Epstein for the metallographic work. The cooperation of the chemistry division of the Bureau of Standards in making the chemical analyses is likewise acknowledged.

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PART 2

PROPOSED SPECIFICATION[6] FOR CAST-IRON CYLINDER PARTS FOR LOCOMOTIVES

I. SCOPE

These specifications cover cast iron for piston-valve bushings, piston-valve packing rings, piston-valve bull rings, cylinder bushings, piston packing rings, and piston-head or bull rings.

II. MANUFACTURE

1. PROCESS

The castings shall be made from good quality close-grained iron.

III. CHEMICAL PROPERTIES AND TESTS

1. CHEMICAL COMPOSITION

Drillings taken from the fractured ends of the transverse-test bars shall conform to the following limits in chemical composition:

	Per cent
Phosphorus, not over	0.70
Sulphur, not over	.12

IV. PHYSICAL PROPERTIES AND TESTS

1. TRANSVERSE TEST

When placed horizontally upon supports 12 inches apart and tested under a centrally applied load, the arbitration-test bars specified in Section IV–3*a* shall show an average transverse strength of not less than 3500 pounds for castings one-half inch or less in thickness, and an average transverse strength of not less than 3800 pounds for castings over one-half inch in thickness; and the deflection for either thickness of material shall not be less than 0.11 inch. The rate of application of the load shall be from 20 to 40 seconds for a deflection of 0.10 inch.

2. CHILL TEST

Before pouring, a sample of the iron shall be taken and chilled in a cast-iron mold, as specified in Section IV–3*b*. The sample shall be allowed to cool in the mold until it is dark red or almost

[6] Prepared at the request of the U. S. Railroad Administration to embody the results of this and other investigations.

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black, when it may be knocked out and quenched in water. The sample, on being broken, must show a close-grained gray iron, with a well-defined border of white iron at the bottom of the fracture. The depth of the white iron must not be less than one-sixteenth of an inch as measured at the center line, for castings one-half inch or less in thickness, nor less than one-eighth of an inch for castings over one-half inch in thickness.

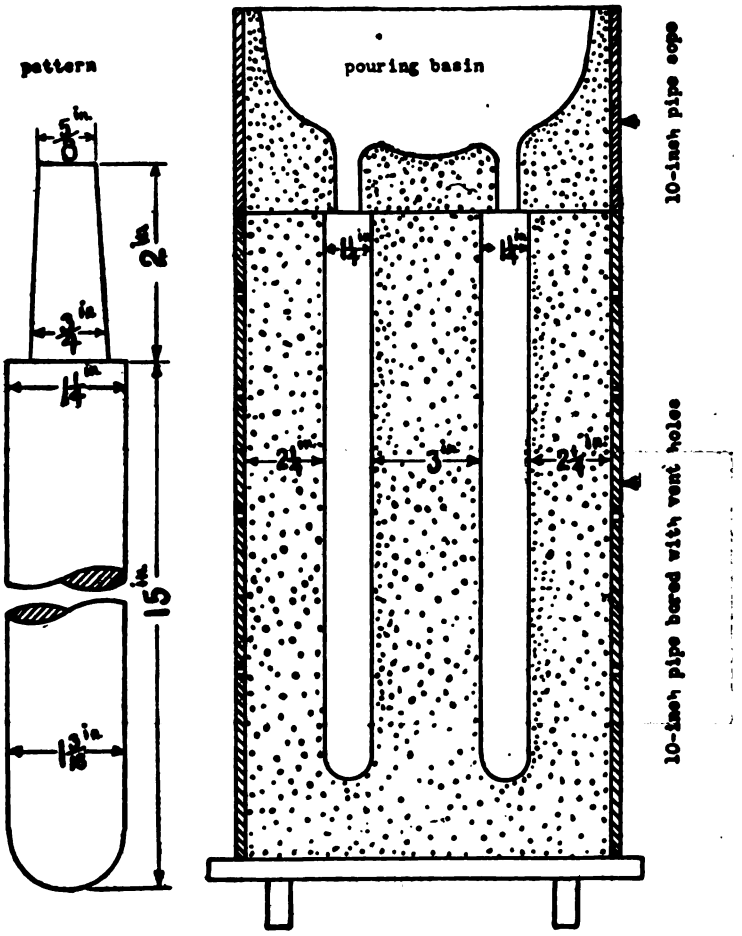

FIG. 9.—*Mold for arbitration test bar*

3. MOLDS FOR TEST SPECIMENS

(*a*) ARBITRATION BAR.—The mold for the bars is as shown in Fig. 9. The bottom of the bar is one-sixteenth of an inch smaller in diameter than the top to allow for draft and strain of pouring. The diameter of the test bar shall not vary more than 0.02 inch above or below the specified size. The flask is to be rammed up

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with green molding sand, a little damper than usual, well mixed and put through a No. 8 sieve, with a mixture of 1 to 12 bituminous facing. The mold shall be rammed evenly and fairly hard, thoroughly dried, and not cast until it is cold. The test bar shall not be removed from the mold until it is cold enough to be handled. It shall not be rumbled or otherwise treated, being simply brushed off before testing.

(*b*) Chill Test.—The form and dimensions of the mold shall be in accordance with Fig. 10.

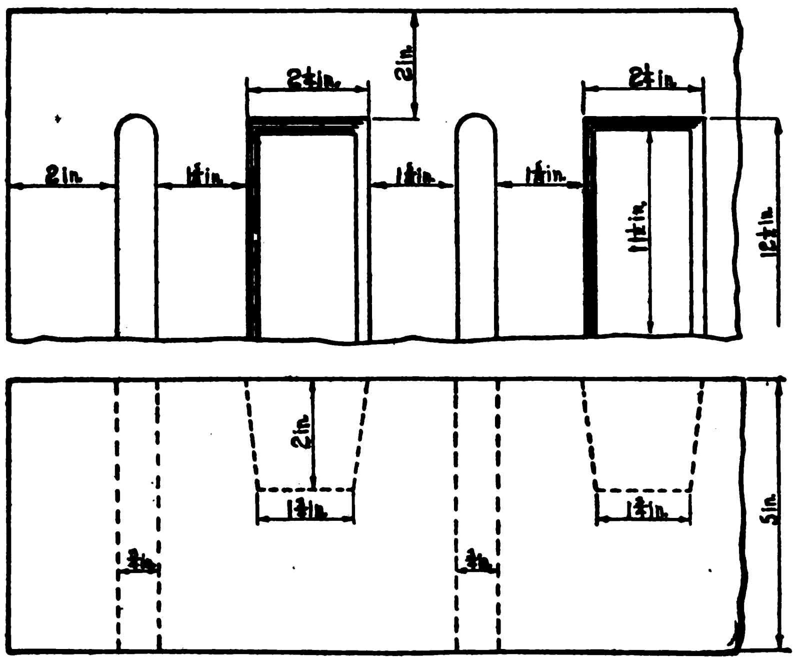

Fig. 10.—*Mold for chill-test specimen*

4. NUMBER OF TESTS

(*a*) Two arbitration-test bars, cast as specified in section IV–3*a*, shall be poured from each ladle used for one or more castings.

(*b*) One chill test, cast as specified in Section IV–3*b*, shall be poured from each ladle of metal used for one or more castings. The chill-test specimens may be cast in adjacent molds, but in such cases a space must be provided between the molds. (See Fig. 10.)

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V. WORKMANSHIP, AND FINISH

1. CHARACTER OF CASTINGS

Castings shall be smooth, well cleaned, free from shrinkage cracks and from other defects sufficiently extensive to impair their value, and must finish to blue-print size.

2. MARKING

Each casting when so specified shall have cast on it, in raised letters and figures, marks designating the maker, the date of casting, the serial and pattern numbers, and other marks specified by the purchaser.

VI. INSPECTION AND REJECTION

1. INSPECTION

(*a*) The purchaser, or his inspector, shall be given a reasonable opportunity to witness the pouring of the castings and test specimens, as well as to be present when the mechanical tests are made.

(*b*) In case the inspector is not present to witness the pouring of the castings and test specimens, the manufacturer will hold the latter for test by the inspector upon his arrival.

(*c*) Inspection shall be made at the place of manufacture, and the manufacturer shall also furnish the facilities for making the mechanical tests, if desired.

2. REJECTION

Unless otherwise specified, any rejection based on tests made in accordance with Section IV–1 shall be reported within five days from receipt of samples.

WASHINGTON, April 30, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 173

TESTS OF BOND RESISTANCE BETWEEN CONCRETE AND STEEL

BY

W. A. SLATER, Engineer Physicist
Bureau of Standards

F. E. RICHART, Structural Engineer
G. G. SCOFIELD, Structural Engineer
Emergency Fleet Corporation

NOVEMBER 1, 1920

PRICE, 25 CENTS

Sold only by the Superintendent of Documents, Government Printing Office
Washington, D. C.

WASHINGTON
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1920

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TESTS OF BOND RESISTANCE BETWEEN CONCRETE AND STEEL

By W. A. Slater, F. E. Richart, and G. G. Scofield

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I. INTRODUCTION

1. PURPOSE AND SCOPE

This paper embodies the results of three somewhat dissimilar investigations which were made by the concrete ship section of the Emergency Fleet Corporation on the bond resistance or adhesion between concrete and reinforcing steel. One investigation was made to study the effect on bond resistance of the applications of various anticorrosive coatings on reinforcement. The second investigation was made to study the length of lap required for effective splicing of reinforcing bars in regions of high tensile stress. The third investigation was made to study the relative merits of different methods of anchoring the ends of stirrups to meet certain conditions which arise in concrete ship construction.

One of the first problems which presented itself in the design of reinforced concrete ships was to determine whether the reinforcement in the shell of a ship would be subject to corrosion, and if so, to devise methods of preventing such action. Corrosion would be dangerous because of the bursting of the concrete due to the formation of a bulky iron compound on the surface of reinforcing

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bars rather than because of injury to the reinforcement itself. Experience with existing reinforced concrete structures exposed to the action of sea water or sea air has shown that a thin covering of ordinary concrete can not be expected to protect steel from corrosion indefinitely.[1] It was anticipated that this action might be prevented in a number of ways, as, for instance, (1) by using a waterproofing paint on the surface of the concrete; (2) by increasing the waterproofing qualities of the concrete itself; or (3) by using some form of protective coating on the reinforcement.

With regard to the first investigation, it was considered probable that the use of a coating on reinforcing bars would affect the bond resistance between the concrete and the bars. Hence, when tests were made to determine the effectiveness of various coatings in preventing the corrosion of steel, tests were also made to determine the effect of the same coatings on the bond resistance which could be developed. These tests were made on pull-out specimens and will be described in Section II of this paper.

The second investigation described in this paper was made because the designer needs definite knowledge as to the effectiveness of lapped bars as a tension splice. In lapping bands of reinforcement, it is important to know how many bars of a band may be safely lapped at one section, and how the length of embedment should vary with variation in the ratio of the cross-sectional area of the lapped bars to the total cross-sectional area of the steel. This is essentially a question of bond resistance, since it involves intensity and distribution of bond stresses along the length of embedment of the lapped bar.

Most of the available data on bond resistance are based on pull-out tests similar to those reported in Section II, in which the concrete block is in compression. Such tests have been shown [2] to agree reasonably well at corresponding slips with bond tests of reinforced concrete beams in which the ends of the reinforcing bars were embedded in a region either of compressive stress or of low tensile stress.

On the other hand, pull-out specimens in which the concrete block was in tension have shown [2] lower bond stresses than those specimens in which the block was in compression, and it seemed probable that bars ending in a region of high tensile stress would develop comparatively low bond stresses. To obtain evidence on

[1] R. J. Wig and L. R. Ferguson, What is the Trouble with Concrete in Sea Water, Eng. News Record, Sept. 20 to Oct. 18, 1917.

[2] D. A. Abrams, Tests of Bond Between Concrete and Steel, Bull. 71, Eng. Exp. Sta., Univ. of Illinois.

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this point test beams were made in which the tension reinforcement was lapped within a region of constant bending moment. The results of these tests are presented in Section III.

In the case of the concrete tank ship, the horizontal pressure of the cargo tends to force the shell outward from the frame, thereby causing tension in the stirrups which extend from the frame into the shell. The proper anchorage of the stirrups in a thin shell is a difficult problem, and it was to furnish information on this subject that the third investigation, as described in Section IV, was carried out.

2. ACKNOWLEDGMENT

The pull-out tests discussed in Section II were made at the concrete laboratory of the Bureau of Public Roads, at Arlington, Va. The tests were made between April, 1918, and January, 1919. A. T. Goldbeck, engineer of tests of the Bureau of Public Roads, was in direct charge of the investigation. R. Harsh and W. E. Rosengarten made most of the tests. The cooperation of the Bureau of Public Roads and of these men personally was of great value in the carrying out of the research work of the concrete ship section.

The beam tests of Section III were made at the John Fritz Laboratory of the Civil Engineering Department, Lehigh University, in December, 1918. Acknowledgment is made to Prof. F. P. McKibben and other members of the faculty for the facilities provided and for their interest in the work.

The pull-out tests of stirrups discussed in Section IV were made in the testing laboratory at the University of Pennsylvania in October, 1919, under the direction of S. C. Hollister of the Emergency Fleet Corporation. The facilities of the laboratory were made available for this investigation by arrangement with Prof. H. C. Berry.

Acknowledgment is made to A. S. Merrill, consulting concrete engineer of the Bureau of Standards, who has reviewed this paper. As a result of Mr. Merrill's work many portions of the paper have been modified, and some portions have been entirely reconstructed.

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II. PULL-OUT TESTS WITH BARS TREATED TO PREVENT CORROSION

1. PRELIMINARY

This group of tests was made primarily to study the effect of the presence of anticorrosive coatings upon the bond resistance of the bars. A very complete series of tests made by Abrams[8] on uncoated bars was of much value in determining the type of specimen to be used, the technique of performing the tests, and the elimination of many undesirable variables which might have obscured the information sought.

The pull-out type of specimen used here has many advantages when it is desired to study a large number of variables. The specimen is easily and accurately made to any desired dimensions, can be tested quickly in a small testing machine, is fairly cheap, and gives results in usual terms with little calculation. For a comparative study like that reported here, it is an excellent type of specimen.

2. DESCRIPTION OF SPECIMEN

The pull-out specimen adopted for this work consisted of a one-half-inch steel reinforcing bar, 24 inches long, embedded axially in a concrete cylinder 6 inches high and 6 inches in diameter. In the position of molding and again in testing, the bar projected about one-half inch above the top of the cylinder and 17½ inches below the bottom.

In a total number of about 350 specimens, 18 kinds of protective coating were investigated. With a few exceptions, a set of 16 specimens was made with each kind of coating, and also with uncoated bars. In each set of specimens, 8 were made with plain square bars and 8 with deformed bars. Each group of 8 specimens was in turn divided into 4 pairs, which were stored under as many different conditions. Plain bars, except in series involving the use of metallic coatings, were painted by hand with a brush, while deformed bars were coated usually by dipping them into the paint. To supplement information obtained from these specimens, additional specimens were made in order to study the effect of varying the manner of application of certain paints.

[8] Bull. 71, Eng. Exp. Sta., Univ. of Illinois.

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3. ANTICORROSIVE COATINGS

A list of the coatings used on the reinforcing bars in these tests is presented in Table 1. This table furnishes the key to the system of symbols used for distinguishing all of the test specimens. A complete specimen number indicates the series number of this investigation in the research program of the concrete ship section, the kind of coating, the kind of bar used and method of applying the coating, and the method of storage. For example, 8HA1 is the specimen number given to a pull-out specimen in series 8, in which an asphaltic coating, H, was applied to a plain square bar by means of a brush, after which the specimen 1 was stored in damp sand for 28 days before testing.

TABLE 1.—System of Reference Symbols Used to Indicate the Nature of the Coatings, Methods of Application, and Conditions of Storage

[Investigation series No. 8]

Kind of coating		Method of applying coating			Method of storage	
First reference letter	Composition	Second reference letter	Kind of bar	How applied	Final reference numeral	Conditions
A	No coating used.	A	Plain square	Painted with brush.	1 and 2	Damp sand, 28 days.
B	Coal-tar pitch, coal-tar naphtha.	B	Deformed square	Dipped.	3 and 4	Damp sand, 1 month, salt water, 5 months.
C	Asphalt paint.	C	do	Painted with brush.	5 and 6	Damp sand, 1 month; salt water and air, 5 months.
D	Do.	D	Plain square	Painted and sanded.	7 and 8	Damp sand, 1 month; air, 5 months.
E	Iron oxide, siliceous material, varnish, turpentine.	E	do	Dipped.		
F	Asphalt paint.					
G	Do.					
H	Do.					
I	Iron oxide, siliceous material, varnish, mineral spirits.					
J	Red lead, linseed oil dryer.					
K	Iron oxide, calcium sulphate, siliceous material, varnish, mineral spirits.					
L	Iron oxide, siliceous material, varnish, mineral spirits.					
M	Asphalt gilsonite paint.					
N	Coal-tar drying oil, petroleum distillate, turpentine.					
O	Zinc (galvanized).					
P	Zinc (sherardized).					
Q	Zinc (metal spray).					
V	Phosphate coating.					
X	Electrogalvanized.					

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Table 2 indicates schematically the exact classification of all specimens tested.

TABLE 2.—Classification of Pull-Out Test Specimens

[The numerals indicate the number of specimens tested]

Reference letter	Coating: Kind	Storage: Damp sand, 28 days, Specimens 1 and 2					Damp sand, 1 month, Salt water, 5 months, Specimens 3 and 4					Damp sand, 1 month, Salt water and air, 5 months, Specimens 5 and 6					Damp sand, 1 month, Air, 5 months, Specimens 7 and 8				
		Plain painted	Deformed dipped	Deformed painted	Plain painted and sanded	Plain dipped	Plain painted	Deformed dipped	Deformed painted	Plain painted and sanded	Plain dipped	Plain painted	Deformed dipped	Deformed painted	Plain painted and sanded	Plain dipped	Plain painted	Deformed dipped	Deformed painted	Plain painted and sanded	Plain dipped
		A	B	C	D	E	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E
A	(No coating used)	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	..
B	Coal tar	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	..
C	Asphaltic	2	2	2	2	...	2	2	2	2	...	2	2	2	2	...	2	2	2	2	..
D	.....do	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
E	Ferric oxide	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	..	...	...
F	Asphaltic	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	..	...	...
G	.....do	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	..	...	...
H	.....do	2	2	2	2	...	2	2	2	2	...	2	2	2	2	...	2	2	2	2	...
I	Ferric oxide	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
J	Red lead	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
K	Ferric oxide	2	...	...	2	2	2	...	...	2	2	2	...	...	2	2	2	...	...	2	2
L	.....do	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
M	Asphaltic	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
N	Coal tar	2	2	...	2	2	2	2	...	...	...	2	2	...	2	2	2	2	...	2	2
O	Galvanized	2	2	...	...	...	2	2	...	...	...	...	...	...	...	...	2	2	...	...	..
P	Sherardized	2	2	...	...	...	2	2	...	...	...	...	...	...	...	...	1	1	...	...	...
Q	Metal spray	2	2	...	...	...	2	2	...	...	...	...	...	...	...	...	2	2	...	...	..
V	Phosphate film	2	2	...	...	...	2	2	...	...	...	...	...	...	...	...	2	2	...	...	..
X	Electro galvanized	2	...	...	...	...	2	...	...	...	...	2	...	...	...	...	2	...	...	...	...

4. MATERIALS

The aggregate used in making the specimens was a Potomac River sand, consisting principally of angular quartz grains and containing a slight amount of mica and clay. It was obtained in Arlington, Va. All of this sand passed a one-fourth-inch screen. A mechanical analysis made with a set of Howard & Morse sieves is presented in Table 3. The average tensile strength of three 1:3 cement mortar briquets was 250 pounds per square inch at seven days, which was 3 per cent higher than the average strength of similar briquets made with standard Ottawa sand.

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TABLE 3.—Mechanical Analysis of Sand

Screen number	Separation size	Per cent passing	Screen number	Separation size	Per cent passing
	Inch			Inch	
¼-inch	0.250	100.0	50-mesh	0.011	24.3
10-mesh	.075	92.3	80-mesh	.0068	11.6
20-mesh	.034	76.1	100-mesh	.0055	7.5
30-mesh	.022	52.0	200-mesh	.0029	2.8
40-mesh	.015	35.2			

Tidewater Portland cement, having a fineness such that 83.9 per cent passed a No. 200 sieve, was used in making the concrete. In the concrete the proportions by weight were 1 part cement, 2 parts sand, and 0.485 parts water. With these proportions the water was 16.2 per cent (by weight) of all dry materials and 75 per cent (by volume) of the cement. Defined by sizes of aggregate, this mix was a mortar instead of a concrete, but, following the usage developed in connection with early concrete ship work, it is here termed concrete.

The steel used consisted of one-half-inch plain square bars and deformed bars of the same nominal cross section and perimeter.[4] The deformed bar was of the form shown in Fig. 1. It had nine lugs per foot of length. The lugs were truncated wedges about 0.05 inch in height, and included an angle of about 40° between the beveled bearing faces. It is evident that this form of lug gave some wedging action in the pull-out test, as verified by the fact that all pull-out specimens made with deformed bars failed by bursting the concrete block. No lateral reinforcement was used to restrain the concrete.

Physical properties of the steel used are given in Table 4.

TABLE 4.—Physical Properties of Steel

Type of bar	Yield point	Ultimate tensile strength	Ultimate elongation in 8 inches
	Lbs./in.²	Lbs./in.²	Per cent
½ inch, plain square	33 150	46 200	14.8
½ inch, deformed	61 300	105 750	16.6

In a few specimens the yield point of the bar was exceeded before failure occurred.

[4] A measurement of one plain bar and one deformed bar showed the perimeter of the deformed bar to be about 4 per cent less than that of the plain bar.

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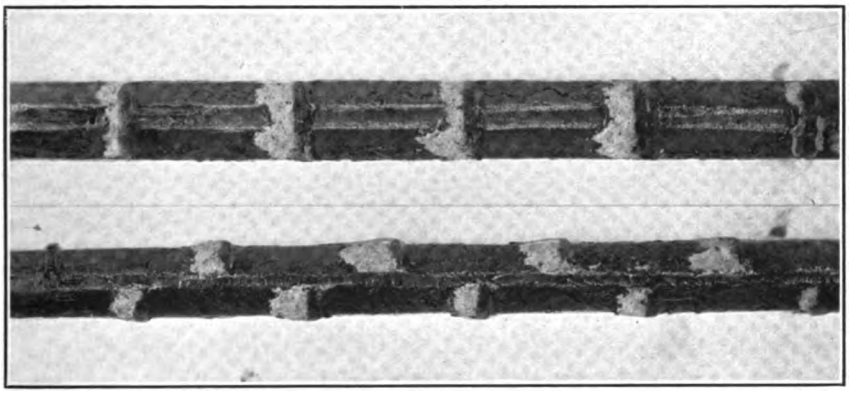

FIG. 1.—*Deformed bar used in pull-out tests*

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5. MAKING OF SPECIMENS

The bars were prepared several days before the molding of the specimens. The plain bars were fairly smooth and had but little rust, and that was removed with emery cloth. One coat of paint was applied with a varnish brush over one-half of the length of each bar painted. The paint was allowed to dry for several days.

The deformed bars had a heavy coating of mill scale and rust. This coating was removed by scraping with the edge of a cold chisel and then brushing with a wire brush. Each bar was held vertically and the lower half of its length was dipped into the paint. The excess paint was allowed to drain off, and the bar was left to dry for several days. All paints were applied cold.

To compare the effect of different methods of applying the coating, a few specimens were made in which the plain bars were dipped and the deformed bars were painted with the brush. A few of the plain painted bars were also coated with a sharp fine sand immediately after painting, to study the effect of such a treatment.

Obviously, the above description does not refer to the application of metallic coatings, which were applied by commercial methods by the manufacturers to whom bars were furnished for such treatment.

In making the specimens, the cylinder molds were placed on a wooden bench. Holes (three-fourths inch in diameter) in the bench permitted the bars to be placed vertically in the axis of the cylinder, with the coated end projecting about one-half inch above the top of the mold. The bars were held in place in the bench by packing cotton waste tightly around them.

The concrete was tamped into place in the mold with a small wooden rod, and the top surface was smoothed off with a trowel.

6. AUXILIARY SPECIMENS

Three compression cylinders, 2 inches in diameter and 4 inches high, were made from the batches of concrete mixed for the pull-out test specimens. Average compressive strengths as determined for the different conditions of age and storage are as follows:

	Lbs./in.^{2}
Damp sand, 28 days	5090
Damp sand, 6 months	6195
Damp sand, 28 days, and sea water, 5 months	7010

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7. STORAGE OF SPECIMENS

The specimens were removed from the molds when 24 hours old and were stored in damp sand in the laboratory for approximately one month. After this preliminary curing, different groups of specimens were accorded different treatment. In all cases specimens were allowed to stand in air 24 hours previous to testing.

Specimens 1 and 2 of each group were removed from the damp sand after 27 days and were tested on the following day. Specimens 3 and 4 were removed from the sand after 1 month and stored in artificial sea water for 5 months, then allowed to dry for 24 hours before testing.

Specimens 5 and 6 were removed from the sand when 1 month old and were stored in artificial sea water and in air alternately 12 hours each day for 5 months and then tested. These specimens were stored on a rack arranged to be readily lowered into and raised from the tank containing the sea water.

Specimens 7 and 8 were removed from the damp sand when 1 month old, stored in the laboratory uncovered for 5 months, and then tested.

All specimens were stored inside the laboratory where the range of temperatures was between 60 and 75° F. There is no indication that these temperatures produced harmful effects on any of the paint coatings.

The storage in sea water was to determine how the bond strength might be affected by the deterioration of paint and metallic coatings and by corrosion of the bars.

The artificial sea water referred to was prepared from the formula given in Table 5, based upon a typical chemical analysis of ocean water.

TABLE 5.—Composition of Artificial Sea Water

Chemical name	Symbol	Percentage composition of solids	Parts by weight
Potassium bromid	KBr	0.29	1
Potassium sulphate	K_2SO_4	2.55	9
Calcium sulphate	$CaSO_4$	3.97	14
Magnesium sulphate	$MgSO_4$	4.12	[a] 14.5
Magnesium chlorid	$MgCl_2$	10.78	38
Magnesium carbonate	$MgCO_2$	.29	1
Sodium chlorid	NaCl	78.00	275
Water (spring)	H_2O	.....	10 000

[a] Anhydrous: equivalent to 29.7 parts of commercial magnesium and sulphate, $MgSO_4.7H_2O$.

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8. METHOD OF TESTING

In making a pull-out test, the specimen was placed on the upper head of the testing machine, as shown in Fig. 2, with the long end of the bar passing down through a central hole in the spherical bearing block. The long end of the bar was engaged in the wedge grips in the moving head of the machine. In a number of cases readings of load were lost, due to the slipping of the bar in the

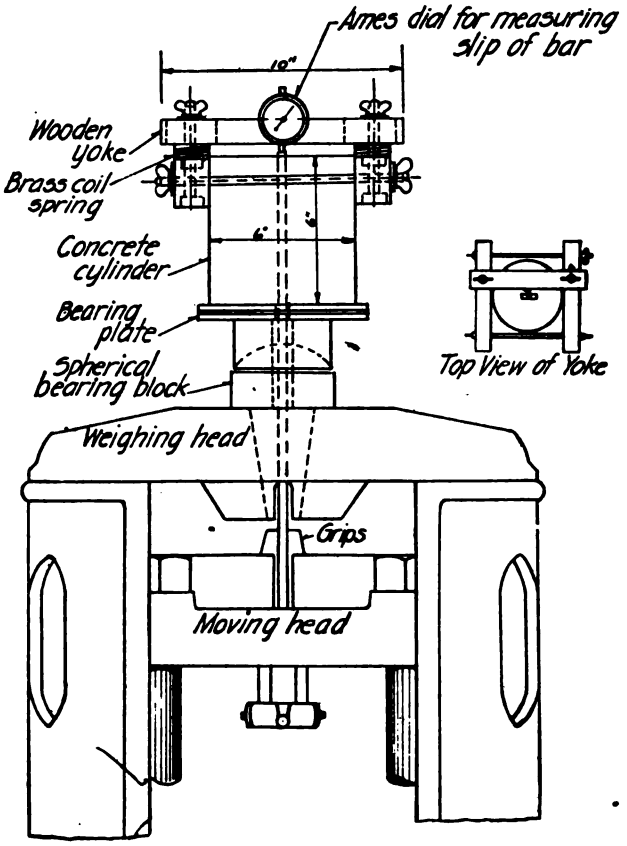

FIG. 2.—*Pull-out specimen in machine in position for testing*

grips at low loads, and in all cases this slipping made it difficult to keep the weighing beam of the machine in balance.[5]

The concrete block was embedded in plaster of Paris on a bearing plate, which transmitted the stress through a spherical bearing block to the weighing head of the machine. The plaster of Paris served to give a uniform bearing between the specimen and the

[5] Since it is particularly desirable to obtain accurate readings of load at the beginning of the slip of the bar, this test could probably be improved by providing a better connection between the bar and pulling head. This could be accomplished by threading the end of the bar and holding it with a nut against a plate bearing against the lower side of the pulling head.

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bearing plate, and the spherical bearing allowed the bar to be adjusted in a vertical position, thus preventing the bending of the bar which otherwise would have occurred when the axis of the bar was not normal to the bearing surface of the concrete block.

The slip of the bar was measured at the upper end of the embedded bar by means of an Ames gage; this was supported by a wooden yoke clamped to the upper part of the concrete block in such a position that the plunger rested on the end of the bar. The dial was graduated so that its smallest division indicated a movement of 0.0001 inch, and a movement of 0.00005 inch was easily estimated. The latter was the smallest slip recorded. Load was applied continuously with the weighing beam kept carefully balanced, and as predetermined amounts of slip were reached, the corresponding load was read and recorded.

The tests were made on a power-driven Olsen testing machine of 20 000 pounds capacity. The slowest speed of the machine was used, giving the pulling head a movement of about 0.07 inch per minute. Due to the slipping of the bar in the grips at low loads, the stretching of the bar, and the compression in the plaster bearing, the slipping, as measured by the Ames gage, was much less than the movement of the pulling head.

9. NATURE OF THE BOND RESISTANCE OF A COATED BAR

A study of bond between concrete and uncoated steel has indicated that bond resistance is made up of two parts, adhesive resistance and sliding resistance. Adhesive resistance comes into play before the bar begins to slip. Sliding resistance is evidently due to friction between the concrete and the surface of the reinforcing bar, which may be, comparatively speaking, rough and irregular in form.

With the deformed bar the projecting lugs provide additional resistance after slipping has begun. It is believed that the adhesive resistance between the concrete and the longitudinal surfaces of the bar is destroyed before there is enough movement to develop much compressive stress between the beveled surface of the lug and the concrete in contact with it.

The action of a coated bar embedded in concrete is undoubtedly of a different nature from that of an uncoated bar. With the more plastic coatings especially, slipping may be due to three possible causes, as follows: (1) Slip between the coating and the concrete; (2) flowing or shearing deformation in the coating in the direction of motion; and (3) slip between the coating and the bar.

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A very soft paint may act merely as a lubricant tending to facilitate slipping of the bar, while a hard, brittle paint may crumble when acted upon by shearing forces. Although the resistance of these paints to distortion is not known, it seems likely that the shearing strength of a paint is less than its adhesion to steel or concrete. On the other hand, a metallic coating, such as zinc, is elastic and has a comparatively high shearing strength, so that failure might be expected to occur through slipping of the zinc on the concrete. A study of the tests shows a considerable variation in the behavior of the different coatings.

10. GENERAL CHARACTERISTICS OF RESULTS

The bond-slip data of all tests have been plotted in Figs. 4 to 38, inclusive, which show the relation between bond stress and slip of bar. Each figure contains the information secured with the use of a certain coating.

The values of bond stress for end slips up to 0.01 inch have been plotted on an extended scale in the left-hand portion of each diagram in order to show distinctly the zone in which end-slip values began to increase rapidly; the same data are included in the right-hand portion of each diagram, which shows values of bond stress for end slips up to 0.20 inch. The points plotted in each diagram represent the average results of tests of two similar specimens. The different conditions of storage have been indicated by the same conventions throughout the diagrams.

The form of the bond-slip curves for coated bars is, in general, similar to that for uncoated bars. A few outstanding differences may be noted briefly. While the plain uncoated bar reached its maximum bond resistance at a slip of 0.01 inch or less, after which the bond stress decreased with further slip, many of the plain painted bars did not reach maximum bond resistance until a slip of 0.03 to 0.05 inch had occurred, and the bond stress remained nearly constant with considerable increase in slip.

However, with some metallic coatings the maximum bond resistance on plain bars was developed at a slip of 0.001 inch and was followed by a sudden large slip of bar with a correspondingly large reduction in bond stress.

Uncoated deformed bars showed an increase in bond resistance with an increase in slip throughout the test. The results for coated deformed bars show two noticeable features: (1) The bond stress was low for small amounts of slip, but the general slope of the bond-slip curves is about the same as for the uncoated bars; (2) in some cases the bond stress developed at the initial slip

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remained constant, or even decreased, until a slip of 0.02 to 0.03 inch was reached, after which the bond-slip curves took on the characteristic slope of the curve for uncoated bars. It is evident that the point at which a curve takes this sudden change in slope corresponds to the slip at which the lugs of the deformed bar have reached a firm bearing against the adjoining concrete.

Tests reported elsewhere[6] have indicated that a very small amount of slip corresponds to critical bond stress in a reinforced concrete beam. For this reason comparisons have been made of bond stresses corresponding to a slip of 0.001 inch and 0.01 inch, respectively. The ratios of these stresses to the maximum bond stresses developed in these tests are given in Table 6 in the form of percentages.

TABLE 6.—Bond Stresses at Specified Slips in Percentage of Maximum Bond Stresses

[Values given are averages for all ages and conditions of storage]

Coating		0.001-inch slip		0.01-inch slip	
Reference letter	Kind	Plain bars	Deformed bars	Plain bars	Deformed bars
		Per cent	Per cent	Per cent	Per cent
A	(No coating used)	94	53	99	62
C	Asphaltic	58	8	91	13
D		60	23	90	36
F		55	15	79	22
G		53	23	86	33
H		74	33	95	44
M		62	26	92	45
	Average	60	21	89	32
B	Coal tar	65	20	88	34
N		68	26	98	37
	Average	67	23	93	36
E	Ferric oxide	67	24	86	38
I		84	33	97	44
K		66	.....	84	.....
L		73	29	95	41
	Average	73	29	90	41
O	Metallic	98	50	80	44
P		100	66	53	45
X		101	.....	74	.....
	Average	100	58	69	45
Q	Metal spray	16	4	76	12
J	Red lead	47	7	66	15
V	Phosphate	92	49	97	56

[6] Abrams, Bull. 71, Eng. Exp. Sta., Univ. of Illinois.

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In order to present graphically a large amount of the information of these tests, Fig. 3 has been prepared. Bond stresses at slips of 0.001 inch and 0.01 inch, and at maximum load are shown. The chart shows that, in general, the bond resistance of coated bars was much less than that for uncoated bars. The dotted horizontal lines drawn in Fig. 3, corresponding to the bond stresses developed in uncoated bars at a slip of 0.001 inch and at the maximum load, enable this comparison to be made readily. Averaging the results for all conditions of storage for each coating, the data of Fig. 3 are summarized and shown as percentages in Table 7.

TABLE 7.—Bond Stresses for Coated Bars in Percentage of Bond Stresses for Uncoated Bars

[Values given are averages for all ages and conditions of storage]

Coating		Percentage of bond stress for uncoated bars						Average
Reference letter	Kind	0.001-inch slip		0.01-inch slip		Maximum bond stress		
		Plain	Deformed	Plain	Deformed	Plain	Deformed	
A	Uncoated	100	100	100	100	100	100	100
C	Asphaltic	30	9	43	14	47	69	.....
D		29	29	41	37	45	65	.....
F		30	19	40	23	50	67	.....
G		28	29	42	36	49	68	.....
H		38	45	46	50	48	71	.....
M		48	32	66	48	71	65	.....
	Average	34	27	46	35	52	67	43
B	Coal tar	53	25	67	36	76	67	.....
N		62	38	83	47	84	78	.....
	Average	58	32	75	42	80	73	60
E	Ferric oxide	30	35	36	49	42	80	.....
I		45	35	48	40	49	56	.....
K		37	.....	43	.....	52	.....	.....
L		40	38	48	47	50	71	.....
	Average	38	36	44	45	48	69	47
O	Metallic	76	78	58	59	72	82	.....
P		89	94	42	69	80	76	.....
X		97	.....	66	.....	89	.....	.....
	Average	87	86	55	64	80	79	75
Q	Metal spray	1	7	5	17	7	88	21
J	Red lead	6	9	8	16	8	63	18
V	Phosphate	90	107	88	104	91	116	99

11. ACTION OF INDIVIDUAL COATINGS

A brief tabular summary of the characteristic action of the various coatings is given in Table 8. As an additional guide in determining the merits of a given type of coating, the resistance of each coating to salt-spray corrosion tests is also presented.

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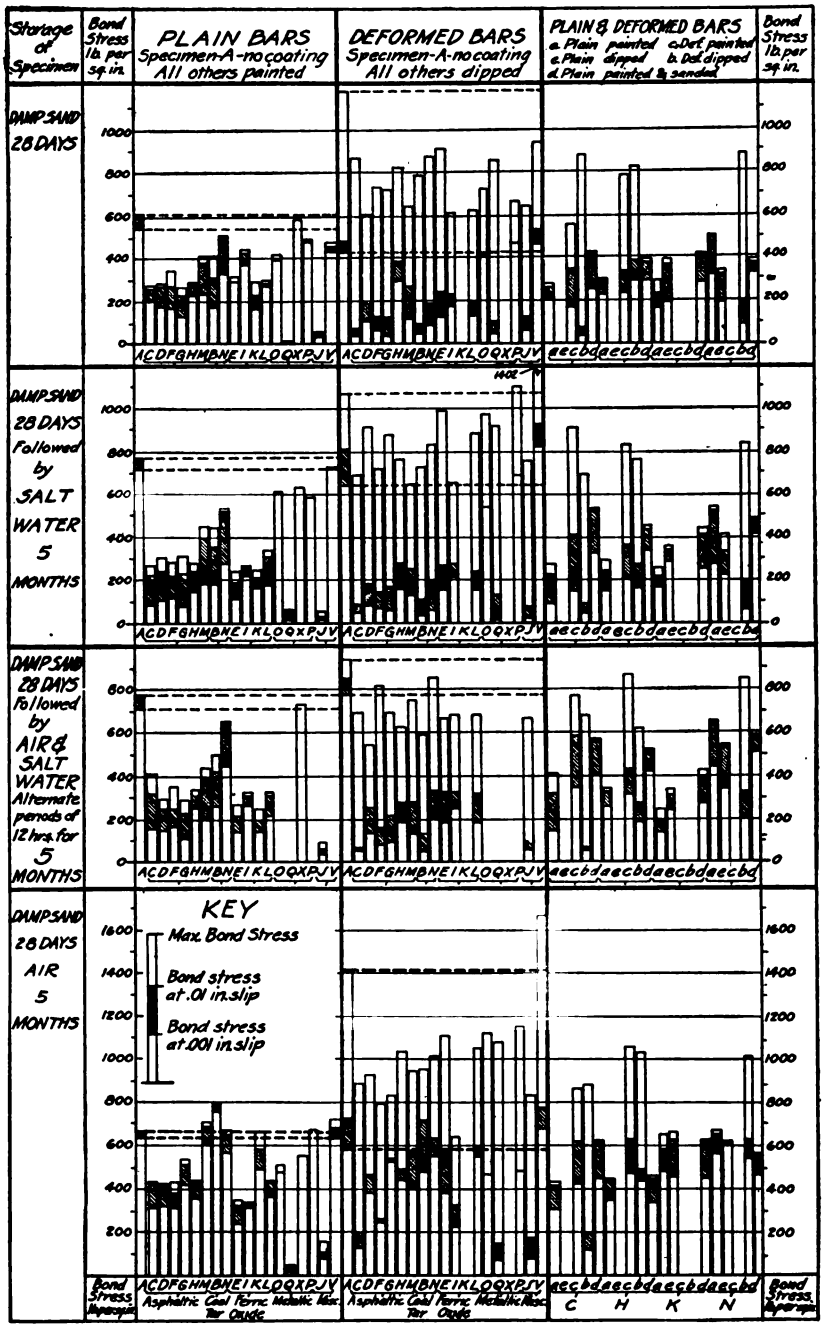
Storage of Specimen
Bond Stress lb. per sq. in.
PLAIN BARS
Specimen-A-no coating
All others painted
DEFORMED BARS
Specimen-A-no coating
All others dipped
PLAIN & DEFORMED BARS
a. Plain painted c. Def. painted
a. Plain dipped b. Def. dipped
d. Plain painted & sanded
DAMP SAND 28 DAYS
DAMP SAND 28 DAYS followed by SALT WATER 5 MONTHS
DAMP SAND 28 DAYS followed by AIR & SALT WATER Alternate periods of 12 hrs. for 5 MONTHS
DAMP SAND 28 DAYS AIR 5 MONTHS
KEY
Max. Bond Stress
Bond stress at .01 in. slip
Bond stress at .001 in. slip
A C D F G H M B N E I K L O Q X P J V
a e c b d a e c b d a e c b d a e c b d a e c b d
C H K N
Asphaltic Coal Tar
Ferric Oxide
Metallic Mix.

FIG. 3.—*Bond stresses developed with different coatings under various conditions of storage*

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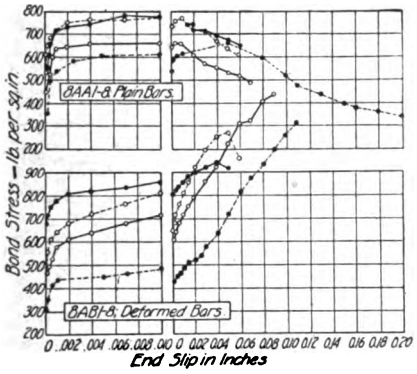

FIG. 4.—*Bond-slip diagrams for uncoated bars*

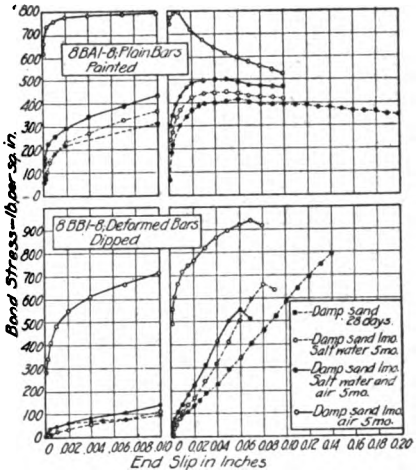
8BA1-8; Plain Bars Painted
8BB1-8; Deformed Bars Dipped
Damp sand 28 days.
Damp sand 1 mo. Salt water 5 mo.
Damp sand 1 mo. Salt water and air 5 mo.
Damp sand 1 mo. air 5 mo.
Bond Stress – lb. per sq. in.
End Slip in Inches

FIG. 5.—*Bond-slip diagrams for coating B; coal tar*

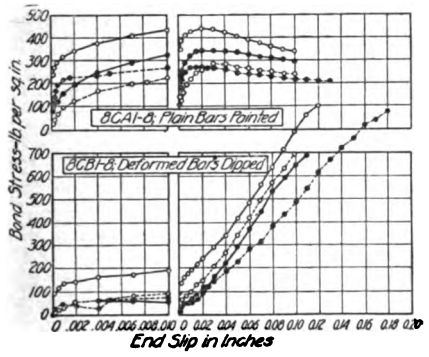

FIG. 6.—*Bond-slip diagrams for coating C; asphaltic*

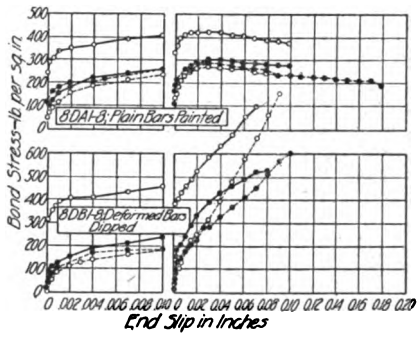

FIG. 7.—*Bond-slip diagrams for coating D; asphaltic*

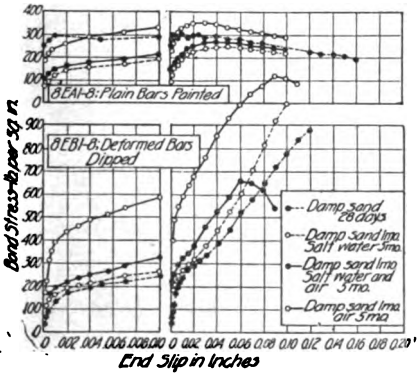
8EA1-8; Plain Bars Painted
8EB1-8; Deformed Bars Dipped
Damp sand 28 days
Damp sand 1 mo. Salt water 5 mo.
Damp sand 1 mo. Salt water and air 5 mo.
Damp sand 1 mo. air 5 mo.
Bond Stress – lb. per sq. in.
End Slip in Inches

FIG. 8.—*Bond-slip diagrams for coating E; ferric oxide*

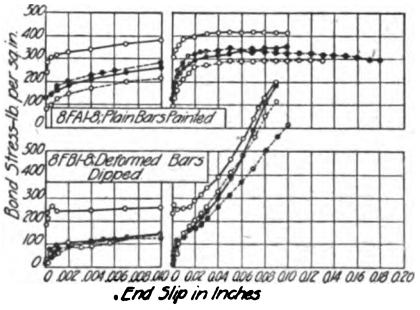

FIG. 9.—*Bond-slip diagrams for coating F; coal tar*

NOTE FOR FIGS. 4 TO 38, INCLUSIVE.—**In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.**

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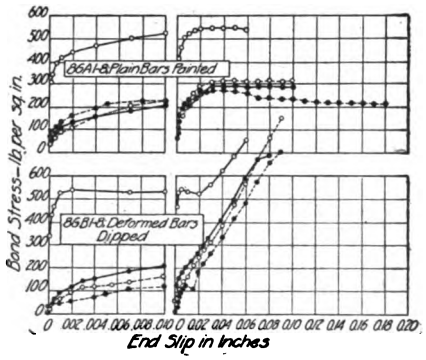

FIG. 10.—*Bond-slip diagrams for coating G; asphaltic*

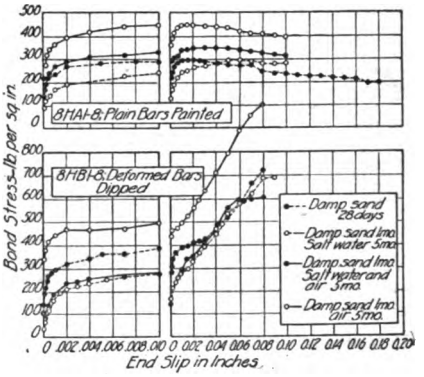
8HAI-8; Plain Bars Painted
8HBI-8; Deformed Bars Dipped
Damp sand 28 days
Damp sand 1mo. Salt water 5mo.
Damp sand 1mo. Salt water and air 5mo.
Damp sand 1mo. air 5mo.
Bond Stress-lb. per sq. in.
End Slip in Inches

FIG. 11.—*Bond-slip diagrams for coating H; asphaltic*

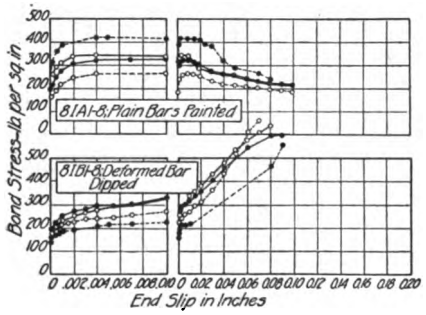

FIG. 12.—*Bond-slip diagrams for coating I; ferric oxide*

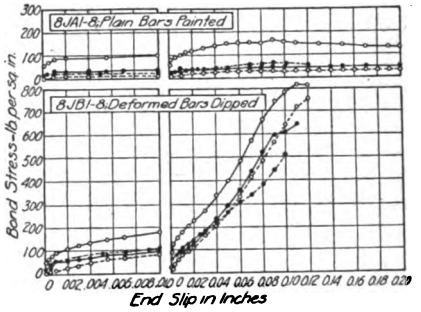

FIG. 13.—*Bond-slip diagrams for coating J; red lead*

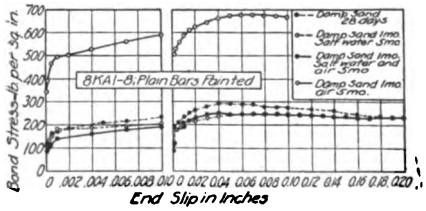
8KAI-8; Plain Bars Painted
Damp, Sand 28 days
Damp Sand 1mo. Salt water 5mo
Damp Sand 1mo. Salt water and air 5mo.
Damp Sand 1mo. air 5mo.
Bond Stress-lb. per sq. in.
End Slip in Inches

FIG. 14.—*Bond-slip diagrams for coating K; ferric oxide*

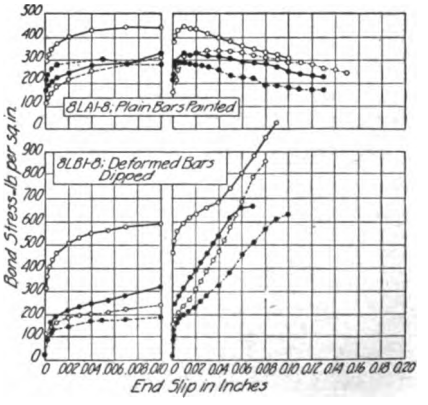

FIG. 15.—*Bond-slip diagrams for coating L; ferric oxide*

NOTE FOR FIGS. 4 to 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

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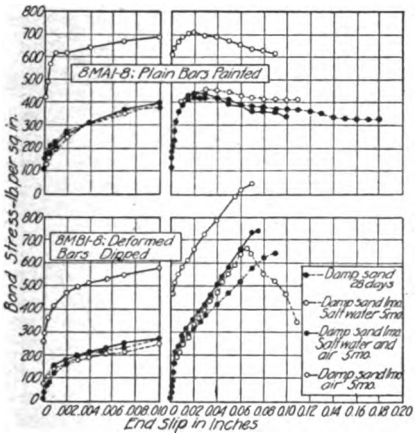
8MA1-8: Plain Bars Painted
8MB1-8: Deformed Bars Dipped
Damp sand 28 days
Damp sand 1mo. Salt water 5mo.
Damp sand 1mo. Salt water and air 5mo.
Damp sand 1mo. air 5mo.
Bond Stress-lb per sq in.
End Slip in Inches

FIG. 16.—*Bond-slip diagrams for coating M; asphaltic*

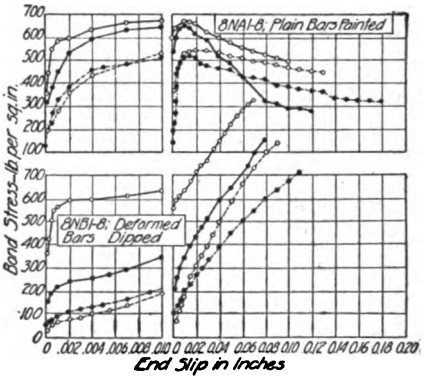

FIG. 17.—*Bond-slip diagrams for coating N; coal tar*

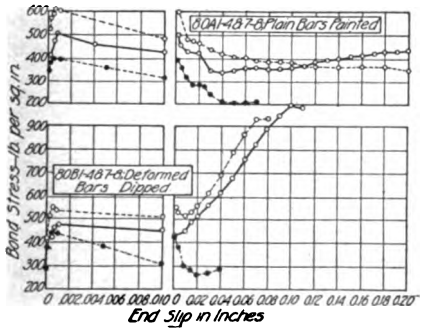

FIG. 18.—*Bond-slip diagrams for coating O; galvanized*

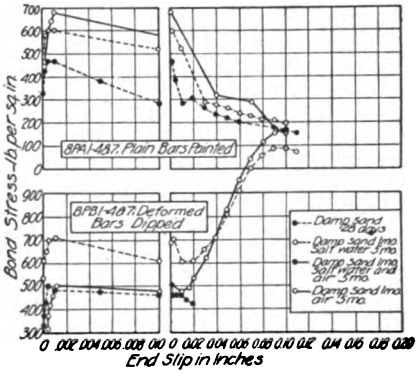
8PA1-4&7: Plain Bars Painted
8PB1-4&7: Deformed Bars Dipped
Damp sand 28 days
Damp sand 1mo. Salt water 5mo.
Damp sand 1mo. Salt water and air 5mo.
Damp sand 1mo. air 5mo.
Bond Stress-lb per sq in.
End Slip in Inches

FIG. 19.—*Bond-slip diagrams for coating P; sherardized*

NOTE FOR FIGS. 4 to 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

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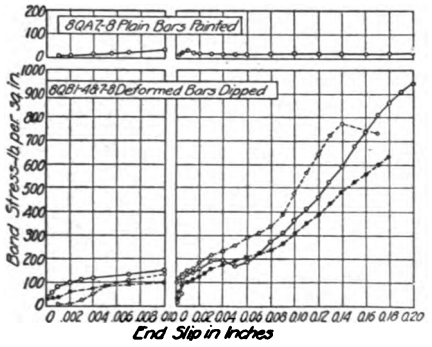

FIG. 20.—*Bond-slip diagrams for coating Q; metal spray*

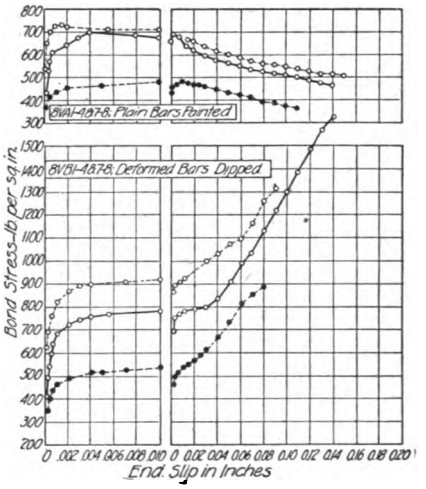

FIG. 21.—*Bond-slip diagrams for coating V; phosphate film*

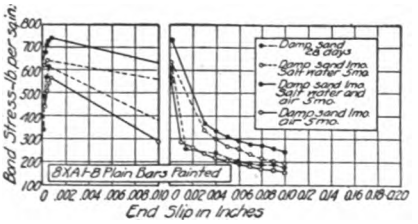
Damp sand 28 days
Damp sand 1mo. Salt water 5mo.
Damp sand 1mo. Salt water and air 5mo.
Damp sand 1mo. Air 5mo.
8XA1-8 Plain Bars Painted
Bond Stress-lb per sq in
End Slip in Inches

FIG. 22.—*Bond-slip diagrams for coating X; electro galvanized*

NOTE FOR FIGS. 4 to 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

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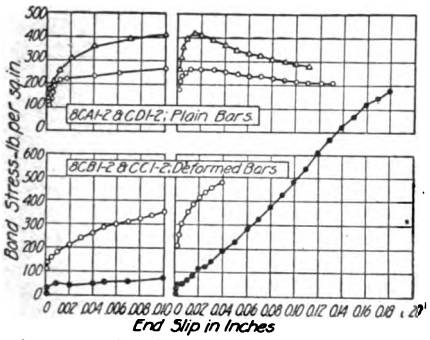

FIG. 23.—*Coating C, stored in damp sand 28 days*

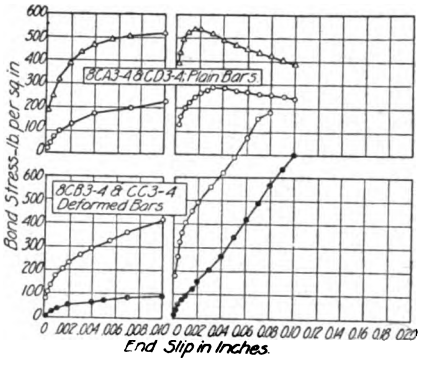

FIG. 24.—*Coating C, stored in damp sand one month, salt water five months*

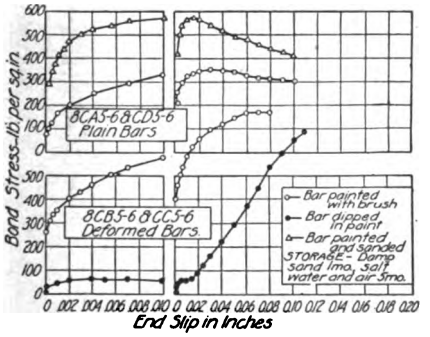
Bond Stress-lb per sq in.
8CA5-6 & CD5-6 Plain Bars
8CB5-6 & CC5-6 Deformed Bars.
Bar painted with brush
Bar dipped in paint
Bar painted and sanded
STORAGE - Damp Sand 1mo, salt water and air 5mo.
End Slip in Inches

FIG. 25.—*Coating C, stored in damp sand one month, salt water and air five months*

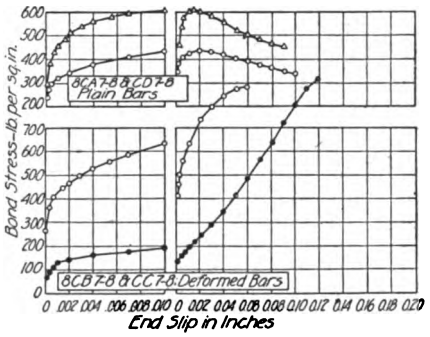

FIG. 26.—*Coating C, stored in damp sand one month, air five months*

NOTE FOR FIGS. 4 TO 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

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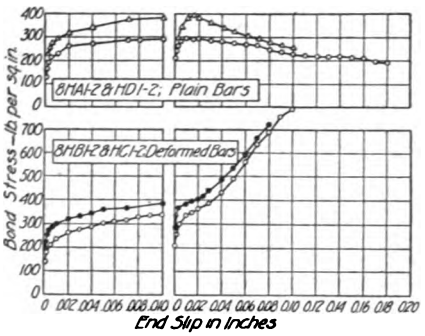

FIG. 27.—*Coating H, stored in damp sand 28 days*

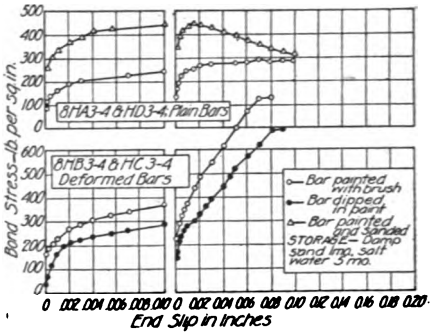
Bond Stress-lb per sq in.
8 HA3-4 & HD3-4 Plain Bars
8 HB3-4 & HC3-4 Deformed Bars
Bar painted with brush
Bar dipped in paint
Bar painted and sanded
STORAGE—Damp sand Imp. Salt water 5 mo.
End Slip in Inches

FIG. 28.—*Coating H, stored in damp sand one month, salt water five months*

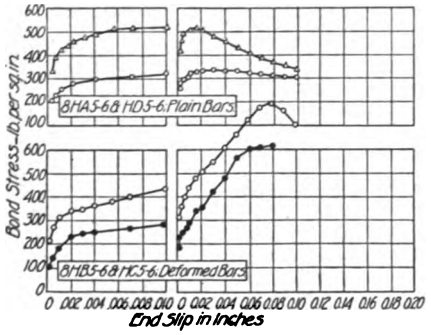

FIG. 29.—*Coating H, stored in damp sand one month, salt water and air five months*

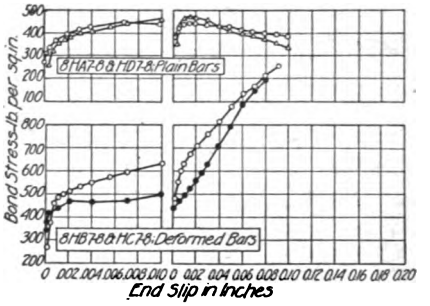

FIG. 30.—*Coating H, stored in damp sand one month, air five months*

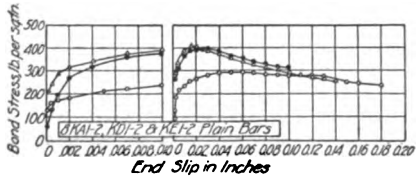

FIG. 31.—*Coating K, stored in damp sand 28 days*

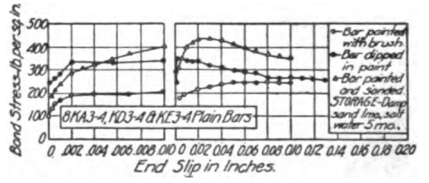
Bond Stress-lb per sq in.
Bar painted with brush
Bar dipped in paint
Bar painted and sanded
STORAGE-Damp sand Imp. salt water-5 mo.
8 KA3-4, KD3-4 & KE3-4 Plain Bars
End Slip in Inches.

FIG. 32.—*Coating K, stored in damp sand one month, salt water five months*

NOTE FOR FIGS. 4 TO 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

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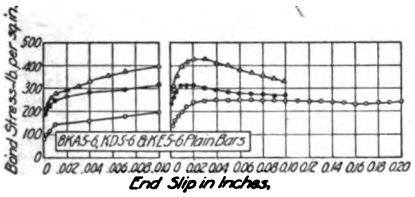

FIG. 33.—*Coating K, stored in damp sand one month, salt water and air five months*

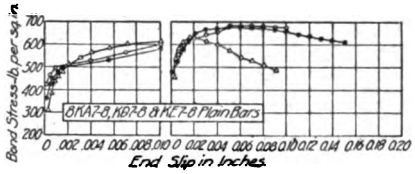

FIG. 34.—*Coating K, stored in damp sand one month, air five months*

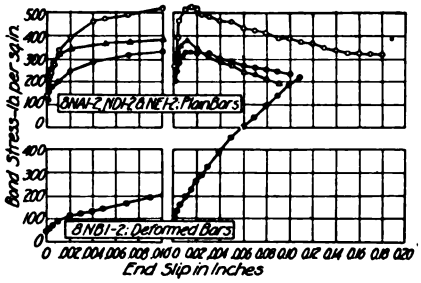

FIG. 35.—*Coating N, stored in damp sand 28 days*

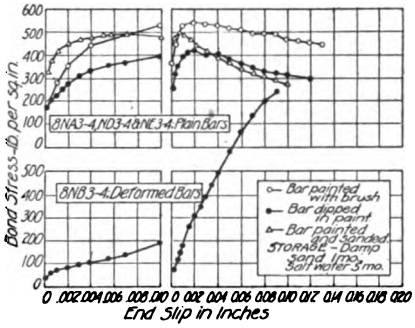
Bond Stress-lb per sq. in.
8NA3-4, ND3-4 & NE3-4 Plain Bars
8NB3-4 Deformed Bars
Bar painted with brush
Bar dipped in paint
Bar painted and sanded
STORAGE - Damp sand 1 mo., Salt water 3 mo.
End Slip in Inches

FIG. 36.—*Coating N, stored in damp sand one month, salt water five months*

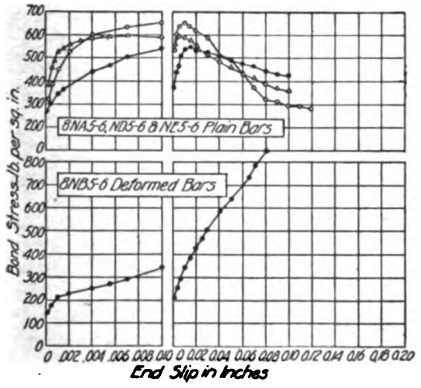

FIG. 37.—*Coating N, stored in damp sand one month, salt water and air five months*

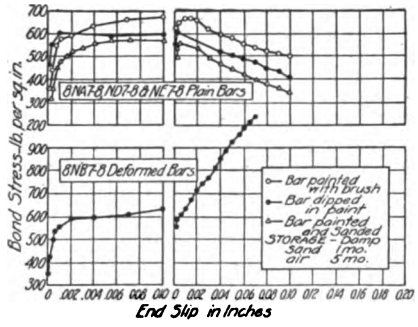
Bond Stress-lb per sq. in.
8NA7-8, ND7-8 & NE7-8 Plain Bars
8NB7-8 Deformed Bars
Bar painted with brush
Bar dipped in paint
Bar painted and sanded
STORAGE - Damp Sand 1 mo., Air 5 mo.
End Slip in Inches

FIG. 38.—*Coating N, stored in damp sand one month, air five months*

NOTE FOR FIGS. 4 TO 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

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TABLE 8.—Characteristic Action of Coatings in Bond Tests and Corrosion Tests

Reference letter	Composition and appearance of coating	Average time of drying		Ratio of bond stress for coated bar to uncoated bar (0.01-inch slip)		Characteristic feature of bond tests	Resistance to salt-spray corrosion test
		Plain	Deformed	Plain	Deformed		
		Days	Days				
B	Coal tar; dried to dull finish, with tough adhesive film.	2	5	0.67	0.36	Bond stress high after 6 months' storage in air; fairly low after other storage; above average on plain bars.	Failed.
C	Asphaltic; surface sticky and not very adhesive.	3	6	.43	.14	Comparatively low, especially on deformed bars; below average of all coatings.	Fair.
D	Asphaltic; fairly hard and tough surface.	3	5	.41	.37	Comparatively low; below average of all coatings.	Do.
E	Ferric oxide; adhesive surface.	2	6	.36	.49	Better result on deformed than on plain bars; about average of all coatings; shows well on 28-day test.	Good.
F	Coal tar; dried to dull finish with tough adhesive film.	2	5	.40	.23	Bond stress high after 6 months' storage in air; fairly low after other storage; above average on plain bars.	Fair.
G	Asphaltic; slightly sticky, glossy and adhesive.	2	6	.42	.36	Slightly below average of all coatings; best after 6 months' storage in air; maximum bond of plain bars continues with considerable slip.	Do.
H	Asphaltic; film sticky and glossy, not very adhesive.	2	6	.46	.50	Slightly above average; paint was scaly after specimen was broken and adhered partly to concrete.	Do.
I	Ferric oxide; hard, dull, rather rough finish.	3	3	.48	.40	Average of all coatings; adhered to concrete of broken specimens; form of bond-slip curves similar to those of uncoated bars.	Good.
J	Red lead; thick, wrinkled film; dried very slowly.	.....	.....	.08	.16	Very poor bond; fair maximum bond on deformed bars; paint adhered to both steel and concrete of broken specimen; appeared chalky.	Excellent.
K	Ferric oxide; smooth, glossy film.	.....	.....	.43	.....	Slightly below average; fair after 6 months' storage in air; maximum bond continued with large slips.	Do.

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TABLE 8—Continued

Reference letter	Composition and appearance of coating	Average time of drying		Ratio of bond stress for coated bar to uncoated bar (0.01-inch slip)		Characteristic feature of bond tests	Resistance to salt-spray corrosion test.
		Plain	Deformed	Plain	Deformed		
		Days	Days				
L	Ferric oxide; paint thick and poor consistency.	5	3	0.48	0.47	Slightly above average; best results with 6 months' storage in air; adhered mainly to concrete in broken specimens.	Failed.
M	Asphaltic; hard, glossy film.	7	4	.66	.48	Better than average; paint appeared slightly brittle in broken specimens.	Fair.
N	Coal tar; film lumpy, not of uniform thickness.	7	4	.83	.47	Considerably above average; bond-slip curves similar to those for uncoated bars; paint adhered to broken specimens.	Failed.
O	Zinc (galvanized); film about 0.003 inch thick.	.....	.....	.58	.59	Much above average; maximum bond at 0.001-inch slip on plain bars followed by sudden slip and decrease of bond; peculiar bond-slip curves for deformed bars.	
P	Zinc (sherardized); film about 0.002 inch thick.	.....	.....	.42	.69	Much above average; maximum bond on plain bars at 0.001-inch slip; action similar to "O."	
Q	Zinc (metal spray); film about 0.001 inch thick.	.....	.....	.05	.17	Very poor bond; a fair maximum bond is developed with deformed bars; practically no bond with plain bars.	
V	Phosphate.....	.....	.....	.88	1.04	Very good bond; action similar to that of uncoated bars; consistently good at all storages.	
X	Zinc (electro galvanized)	.....	.....	.66	.....	Good bond, very much above average; behavior similar to that of coatings "O" and "P."	

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12. COMPARISON OF PLAIN AND DEFORMED BARS

The relation between bond stresses developed by plain bars and those developed by deformed bars has already been shown in the bond-slip curves and in Fig. 3. In order to provide a definite numerical comparison, the information of Table 9 was prepared. Results given for coated bars are the average for all coatings tested.

TABLE 9.—Comparison of Bond Stresses in Plain and Deformed Bars

[Values given are averages for all ages and all conditions of storage]

Treatment of bar	Bond stress at 0.001-inch slip		Ratio, plain to deformed	Bond stress at 0.01-inch slip		Ratio, plain to deformed	Maximum bond stress		Ratio, plain to deformed
	Plain	Deformed		Plain	Deformed		Plain	Deformed	
	Lbs/in.²	Lbs/in.²		Lbs/in.²	Lbs/in.²		Lbs/in.²	Lbs/in.²	
Uncoated	655	610	1.07	704	715	0.99	708	1153	0.62
Coated	288	220	1.31	348	293	1.19	400	836	.48

Conclusions based on the relation of bond stresses for plain and deformed bars, as shown in Table 9, must be qualified by the fact that the method of applying the coatings was different for the plain bars from that used for the deformed bars. This subject is discussed in Section II–14.

13. EFFECT OF VARIATIONS IN CONDITIONS OF STORAGE

Fig. 3 shows in a general way that the bond strength for specimens six months old was considerably higher for those stored in air than for the corresponding specimens stored under other conditions. In order to facilitate comparison, Table 10 has been prepared. This table shows the effect on the bond strength of the various conditions of storage.

With plain bars continuous storage in sea water produced about the same effect as alternate storage in sea water and air. Bond stresses from both of these averaged about three-fourths as high as those developed with the specimens stored in air. The effect of a given method of storage does not show consistent results for the plain and deformed types of bar, but in general no appreciable difference in bond stress may be expected between the two conditions of continuous and intermittent exposure to sea water.

The unprotected ends of bars were much more badly corroded in the case of intermittent exposure to sea water than in the case

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of continuous exposure, but examination of the specimens showed that this condition did not extend to the embedded portion of the bars.

TABLE 10.—Effect of Conditions of Storage on Bond Strength

[Maximum bond stress for various methods of storage in percentage of maximum bond stress for storage in damp sand 28 days and in air 5 months]

Coating		Plain bars			Deformed bars		
Reference letter	Kind.	Damp sand 28 days	Damp sand 28 days Sea water 5 months	Damp sand 28 days Air and sea water 5 months	Damp sand 28 days	Damp sand 28 days Sea water 5 months	Damp sand 28 days Air and sea water 5 months
		Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
A	Uncoated	92	116	116	83	75	66
C	Asphaltic	64	61	79	97	78	77
D		67	64	69	65	98	58
F		80	68	83	92	91	101
G		50	58	54	87	105	83
H		65	63	77	79	74	59
M		59	64	63	68	69	78
	Average	64	63	71	81	86	76
B	Coal tar	51	57	64	83	76	61
N		76	81	97	87	82	84
	Average	64	69	81	85	79	73
E	Ferric oxide	84	74	77	81	89	60
I		128	77	95	95	102	105
K		43	37	37	.....	.....	.....
L		68	78	75	59	84	64
	Average	81	66	71	78	92	76
O	Metallic	81	120	.....	63	87	.....
P		71	89	.....	43	95	.....
X		105	113	129	.....	.....	.....
	Average	86	107	129	53	91	.....
Q	Metal spray	.....	125	.....	72	84	.....
J	Red lead	35	36	56	77	92	80
V	Phosphate	67	103	.....	57	84	.....
	Average (all specimens)	71	78	78	76	86	75

14. EFFECT OF MANNER OF APPLYING COATINGS

In order to determine whether or not the bond strength of coated bars was affected by the manner of applying the coatings, specimens were made with four paints, C, H, K, and N (see Table 2), using three methods of application. These paints represented the asphaltic, ferric oxide, and coal-tar types. The bond-slip curves for this particular study are plotted in Figs. 23 to 38, and the

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information is also shown in Fig. 3. The curves show some contradictory results, in that for cases which are comparable the highest bond resistance was sometimes obtained with painted bars and sometimes with dipped bars.

Table 11 shows relative values of bond stresses at 0.01 inch slip for all methods of applying coatings. For convenience, results from painted bars have been taken as unity. As four paints only were subjected to this comparison, the relations shown in this table should not be regarded as conclusive.

TABLE 11.—Relative Values of Bond Resistance at 0.01-Inch Slip—Effect of Various Methods of Applying Coating

Method of applying coating	"C" asphaltic		"H" asphaltic		"K" ferric oxide	"N" coal tar
	Plain bar	Deformed bar	Plain bar	Deformed bar	Plain bar	Plain bar
Painting	1.00	1.00	1.00	1.00	1.00	1.00
Dipping	.....	0.19	.....	0.84	1.46	0.70
Painting and sanding	1.74	.....	1.45	.....	1.65	.86

15. SUMMARY

Results of this series of tests have shown a wide variation in the behavior of different coatings. A few of the results may be summarized here for convenient reference.

(*a*) The maximum bond stress developed by bars which had been painted was generally considerably less than for unpainted bars, but the reduction in maximum bond stress due to galvanizing and some similar processes was less than that due to painting.

(*b*) With the exception of certain metallic coatings, the maximum bond stress for plain coated bars was reached after considerably greater slip than with uncoated bars. The amount of slip of coated bars at maximum stress was often 0.02 to 0.03 inch. Furthermore, the bond resistance did not decrease as rapidly as with uncoated bars with continued slip after the maximum load had been reached. Certain metallic coatings, however, reached maximum bond resistance with very small slip, followed by a sudden decrease in bond load with increasing slip. In a reinforced concrete beam under a constant load, such a yielding of the bond resistance might cause failure without warning.

(*c*) Coated deformed bars apparently slipped considerably before the corrugations or lugs reached a firm bearing. After this had taken place, the increase of resistance with increase of slip was similar to that in uncoated deformed bars.

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(*d*) The bond resistance at a slip of 0.001 inch was a much smaller proportion of the maximum bond resistance for coated bars than for uncoated bars. A slip of 0.001 inch has been considered by some[7] to produce critical conditions of bond stress in a beam.

(*e*) For pull-out specimens employing coated bars, continuous and intermittent storage in artificial sea water resulted in bond strength averaging about three-fourths as great as from storage in air.

(*f*) Sanding of painted bars tended to increase the bond resistance in a number of cases; also dipping bars in paint often resulted in a lower maximum bond stress than was found for bars painted with a brush. However, the effectiveness of the manner in which a coating is applied seemed to depend upon the nature of the paint, and should be determined by test in every case in which it is important to have this information.

III. TESTS OF REINFORCED CONCRETE BEAMS WITH TENSION BARS LAPPED IN A REGION OF UNIFORM BENDING MOMENT

1. PRELIMINARY

This part of the investigation was made on 4 rectangular beams under two-point loading. Each beam had a one-half-inch reinforcing bar spliced in the region of uniform bending moment. The length of the lap was in all cases 41 inches (82 diameters) to insure a safe splice. The object in making tests with a lap of this amount was to secure data as a basis for recommendation as to the length of lap necessary (1) when continuous and spliced bars are adjacent, and (2) when all bars are spliced at the same section. There are a number of other conditions which should be studied in this kind of investigation, but it was not feasible to make these studies in connection with this work.

2. DESCRIPTION OF SPECIMENS

Four beams, 10 feet 8 inches long, 10 inches wide, and 12 inches deep, were made. In 3 of the beams the tension reinforcement consisted of 6 plain round bars, extending the full length of the beam (here termed through bars), and 2 short bars (here termed lapped bars), extending from opposite ends of the beam to a point 20½ inches beyond the center of the span. This gave the short bars a lap of 41 inches, which was symmetrical about the center line of the span. In the fourth beam no through

[7] Abrams, Bul. 71, Eng. Exp. Sta., Univ. of Illinois.

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bars were present, but the lapped bars were placed in the same relative positions as in the other 3 beams. All lapped bars were one-half inch plain round. All bars were anchored at the ends of the beams by means of semicircular hooks. The lapped bars had no mechanical anchorage at the ends of the laps. The numbers and sizes of bars and the percentage of reinforcement in the several beams are summarized in Table 12.

TABLE 12.—Reinforcement of Beams with Lapped Bars

Beam No.	Through bars		Lapped bars		Reinforcement	Ratio [a] area of through bars to lapped bars
	Number	Diameter	Number	Diameter		
		Inch		Inch	Per cent	
8AAA1	0	.....	2	½	0.2	0
8AAB1	6	½	2	½	1.4	6
8AAC1	6	⅝	2	½	2.1	10.3
8AAD1	6	¾	2	½	2.9	13.5

[a] Only one lapped bar considered effective for determining this ratio.

For convenience in designating gage lines, the bars were numbered consecutively from 1 to 8, beginning with the bar next to

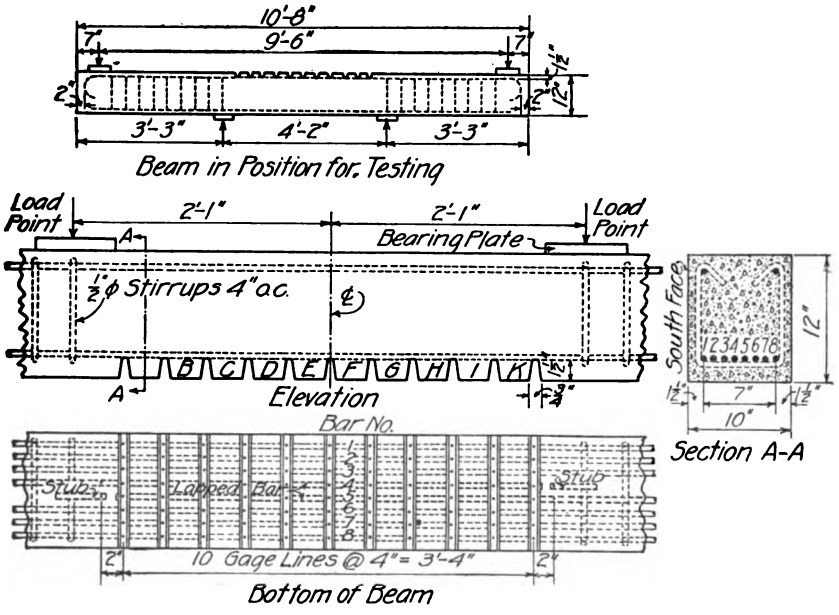

FIG. 39.—*Details of test beams with lapped bars*

the south side of the beam as placed in the testing machine, except for the beam having only the lapped bars. In this case the bars were numbered 4 and 5, the numbers given corresponding

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to the numbers of the lapped bars in all the other beams. Fig. 39 shows the arrangement of the reinforcing bars in each beam. To prevent failure by diagonal tension, nine stirrups, made from one-half-inch plain round bars, were placed near both ends of each beam. Exposure at spots on the tension bars, for the purpose of taking strain gage readings, was provided for by placing wooden blocks in the bottom of the forms across the beams at each row of gage points before pouring. These blocks were about three-fourths inch wide and 1½ inches high and extended the full width of the beam. They were tapered slightly to facilitate removal.

The reinforcing steel used was of the quality known as shell discard. Average results of physical tests are given in Table 13.

TABLE 13.—Physical Properties of Reinforcing Steel Used in the Beam Tests

Diameter	Yield point	Ultimate tensile strength	Elongation in 8 inches	Reduction of area
	Lbs./in.²	Lbs./in.²	Per cent	Per cent
One-half inch	64 000	105 000	16	27
Five-eighths inch	50 000	90 000	21	46
Three-fourths inch	54 000	93 000	20	21

3. MATERIALS

The concrete was mixed in the approximate proportions of 1:1:1, by volume. The quantities of the materials were determined by weight. A rich mix was chosen in order to make the conditions closely similar in this respect to those in concrete ship construction. The fine aggregate was of two sizes: (1) A bank sand, all of which passed a one-eighth-inch screen; and (2) screenings from the gravel varying from one-eighth to one-fourth inch. The coarse aggregate consisted of pebbles from one-fourth to one-half inch in size. Lehigh Portland cement was used, samples of which were tested at the Bureau of Standards, Washington, D. C., and found to conform to the United States Government specifications for Portland cement.[8]

4. MAKING OF SPECIMENS

The blocks of wood used to expose the steel for strain gage readings were tacked to the bottom of the form, and the tension bars were fastened to them. A steel stub, one-half inch in diameter and 4½ inches long, was placed at the unanchored end of each

[8] Circular of the Bureau of Standards, No. 33.

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ample, as with the exception of having readings with the strain gage between the two and between the ends of the middle bar. The stirrups were securely wired to the tension reinforcement, and were held in place at the top of the beam by wiring them to a one-half inch iron round which extended the full length of the beam. The same unit was used for all four beams.

The concrete was mixed in a "Standard" batch mixer of about one cubic foot capacity. Sufficient aggregate for each beam was taken from the storage bin, thoroughly mixed, and its moisture content was determined. Enough water was added to bring the total water content to 9 per cent of the total dry weight of the aggregate. Each batch was mixed for five minutes at the rate of 14 revolutions per minute, turned into a tight box, and carried to the testing frame in the mixer. As the concrete was shoveled into the forms it was tamped, and the form was jarred with rubber mallets. Six 6 by 12 inch cylinders were made with each batch. These were tested at the age of 7 days, and at the same age as the beam when tested. Results of the tests of these specimens are given in Table 4.

TABLE 4.—Strength of Beams and Control Cylinders

Beam No.	Modulus of rupture	Cylinder strength	
		7 days	27 days
	Pounds per square inch	Pounds per square inch	Pounds per square inch
A-1	560	1,930	2,880
A-2	530	1,885	2,965
A-3	510	1,860	2,915
A-4	540	1,890	2,955

Beams 2 years old were tested.

The beams were removed from the forms 24 hours after pouring and were covered with wet burlap, which was sprinkled each day until the beams were removed for the test.

3. METHOD OF TESTING

The tests were made in a 200,000-pound Olsen testing machine. Before placing the specimen in the testing machine holes were drilled in the tension flange for strain gage readings. The sides and bottom of the beam were whitewashed to facilitate the detection of cracks.

At each gage line in the drilled holes a spot of whitewash was placed on the stirrups and also on the concrete immediately adjoining it to aid in the detection of slip of the bar during the

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lapped bar with the intention of taking readings with the strain gage between the stub and the end of the lapped bar. The stirrups were securely wired to the tension reinforcement, and were held in place at the top of the beam by wiring them to a one-half inch plain round bar which extended the full length of the beam. The same form was used for all four beams.

The concrete was mixed in a "Wonder" batch mixer of about 3 cubic feet capacity. Sufficient aggregate for each beam was taken from the storage bin, thoroughly mixed, and its moisture content was determined. Enough water was added to bring the total water content to 13 per cent of the total dry weight of the batch. Each batch was mixed for five minutes at the rate of 12 revolutions per minute, dumped into a tight box, and carried by a traveling crane to the form. As the concrete was shoveled into the form, it was puddled, and the form was jarred with hand hammers. Six 6 by 12 inch cylinders were made with each beam, 3 to be tested at the age of 7 days, and 3 at the same age as the beam when tested. Results of the tests of these specimens are shown in Table 14.

TABLE 14.—Strength of Beams and Control Cylinders

Beam No.	Maximum load [a]	Cylinder strength	
		7 days	27 days
	Pounds	Lbs./in.²	Lbs./in.²
8AAA1	5 600	3195	4830
8AAB1	55 150	3355	5465
8AAC1	51 300	3050	5215
8AAD1	76 400	3040	4575

[a] Beams 28 days old when tested.

The beams were removed from the forms 24 hours after pouring and were covered with wet burlap, which was sprinkled each day until the beams were prepared for the test.

5. METHOD OF TESTING

The tests were made in a 300 000-pound Olsen testing machine. Before placing the specimen in the testing machine, holes were drilled in the reinforcing bars for strain gage readings. The sides and bottom of the beam were whitewashed to facilitate the detection of cracks.

At each gage hole on the lapped bars a spot of whitewash was placed on the exposed bar and on the concrete immediately adjoining it to aid in the detection of slip at the lap during the

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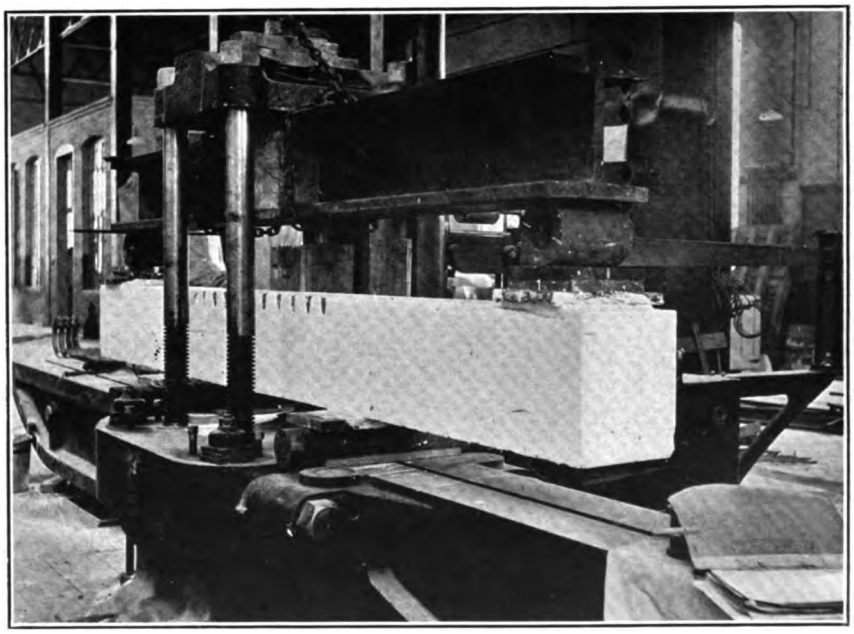

FIG. 40.—*Beam in testing machine ready for application of load*

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test. The same procedure was followed on the through bars at the gage holes nearest the load points. It was found that these whitewashed spots gave a sensitive means of detecting any slip, as a minute movement was unfailingly shown by a crack in the whitewash on the sides of the spot parallel to the direction of slipping.

The beams were placed in the testing machine with the tension side uppermost, and were supported on the base of the machine at two points, each of which was 2 feet 1 inch from the center of the beam. They were loaded at the ends of the span through the medium of an H beam attached to the moving head of the testing machine. Fig. 40 is a view of one of the beams in the testing machine.

Since the points of support in these tests corresponded to the load points of beams tested in the normal position, they will be referred to in this paper as load points. The span was 9 feet 6 inches. Deflections were measured at the center of the beam. Readings of deformation were taken with a Berry strain gage on each of the bars in a series of 10 consecutive gage lines, each 4 inches long, 5 gage lines being taken on each side of the center line of the span. For each gage line on a lapped bar there was a gage line beside it on each through bar.

Sections through a beam bisecting a gage line on each bar were designated by letters from A to K, inclusive, except that letter J was omitted. Any gage line is identified by its section letter and bar number; thus K6 indicates the gage line which is cut by section K of bar No. 6. Bar No. 5 ended near section A, and bar No. 4 near section K, beyond the lettered section in each case.

Three sets of strain gage readings were taken at low loads on each beam to furnish a double check on the accuracy of the zero readings. As the test proceeded, strain gage readings were taken at various increments of load; length and direction of cracks were observed with corresponding loads; and the whitewashed observation points were watched to detect any slip of the lapped bars. As the strain gage readings were taken on both sections of the spliced bar at the lap, they are treated as two bars when discussing the phenomena of the lap, but considered as only one bar effective as tension reinforcement.

6. PHENOMENA OF THE TESTS

(*a*) Beam 8AAA1, poured on November 22, 1918, was set up for testing on December 20. This beam had as tension reinforcement two short one-half-inch plain round bars, lapped 41 inches

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at the center of the beam, and no through bars. These bars were given the numbers 4 and 5 since they corresponded in positions with bars 4 and 5 of the other beams. Strain gage readings were taken at applied loads of 1000, 2000, 3000, 4000, 5000, and 5600 pounds.

Further details: No cracks appeared until a load of 3000 pounds had been applied. At this load a small tension crack 3½ inches long opened up in the northwest quarter, 20¼ inches from the center of the beam. A similar crack in the same relative position showed in the northeast quarter of the beam. A small crack about 1 inch long showed at the center of the beam. Cracking on the south face of the beam was similar to that on the north face. These cracks started from the depressions left in the concrete by the wooden blocks used to expose the reinforcing bars at the gage holes.

At a load of 4000 pounds the crack in the northwest quarter increased slightly in length. On the southeast face several very fine cracks opened up, showing no connection with each other and no tendency to take the same general direction. Each was about 1½ inches long.

At a load of 5000 pounds the crack in the northwest quarter had extended about 10¼ inches into the beam and had increased very much in width. It showed in similar manner in the southwest quarter. A small crack about 4¼ inches long opened up on the northeast face. After about one hour the load, which had fallen off slightly, was brought up to 5000 pounds, and another set of strain gage readings was taken. No new cracks had appeared. Upon increasing the load to 5300 pounds the tension crack at the east load point opened suddenly, and the load fell off slightly with the machine running; but it immediately picked up and increased to 5600 pounds, which was the maximum load attained. The machine was kept running for some time after the maximum load was reached, the beam holding up a load of 3900 pounds.

After removing the beam from the testing machine, an examination showed that bar No. 4 had slipped 0.26 inch at the unanchored end. On this bar all the whitewashed points showed the bond broken between steel and concrete and also indicated considerable movement of the bar. No. 4 showed movement at all of the observation points except the two nearest the unanchored end. These points were three-fourths inch and 4¾ inches, respectively, from the unanchored end. All of the other whitewashed observation points showed a disturbance of the bond by fine cracks in the whitewash parallel to the bar, but whatever movement there was in the bar was too small to measure by any means available.

(*b*) Beam 8AAB1, poured on December 3, was tested on December 31. The tension reinforcement of this beam consisted of six one-half-inch plain round through bars and two one-half-inch plain round short bars lapped 41 inches at the center of the beam. Strain gage readings were taken at loads of 1000, 3000, 5000, 8000, 22 000, 36 000, 50 000, and 55 150 pounds.

Further details: At a 5000-pound load two very fine cracks showed in the southwest quarter and one in the northwest quarter. These cracks were short tension cracks outside of the load points.

At an 8000-pound load small tension cracks appeared in all the quarters, none of which were over 3 inches long. Very fine cracks appeared at the ends of the lapped bars, also at the center of the beam.

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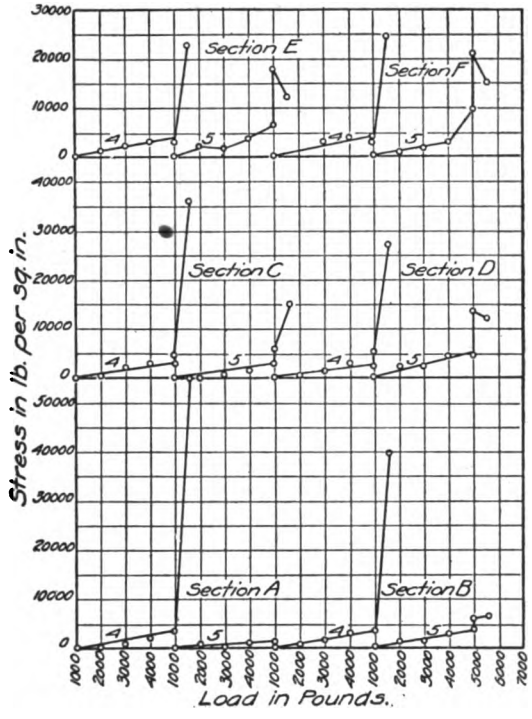

Fig. 41.—*Beam 8AAA1; lapped bars only. Sections A–F*

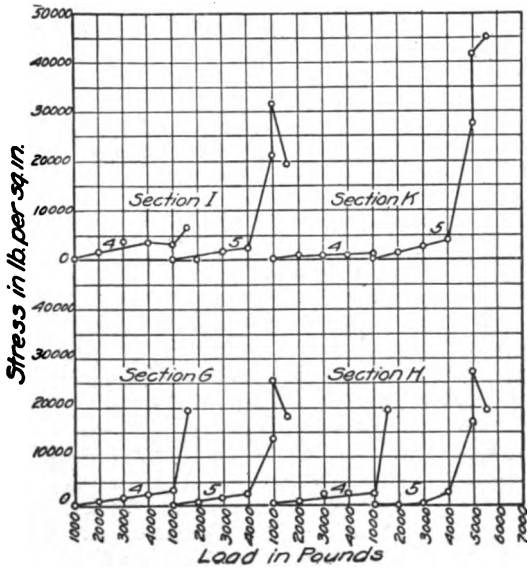

Fig. 42.—*Beam 8AAA1; lapped bars only. Sections G–K*

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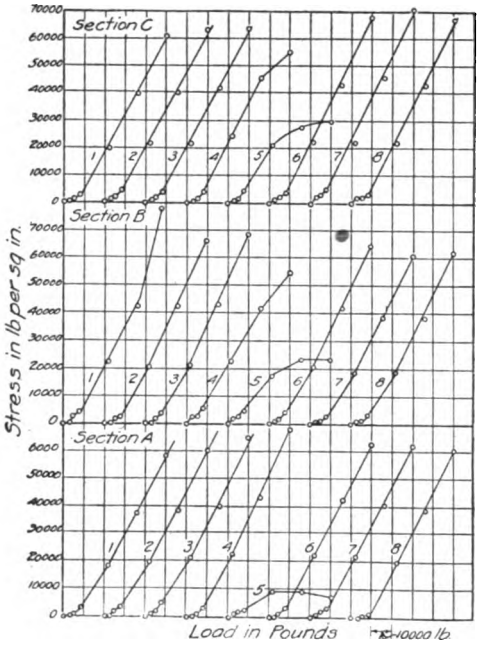

FIG. 43.—*Beam 8AAB1; one-half inch through and one-half inch lapped bars. Sections A–C*

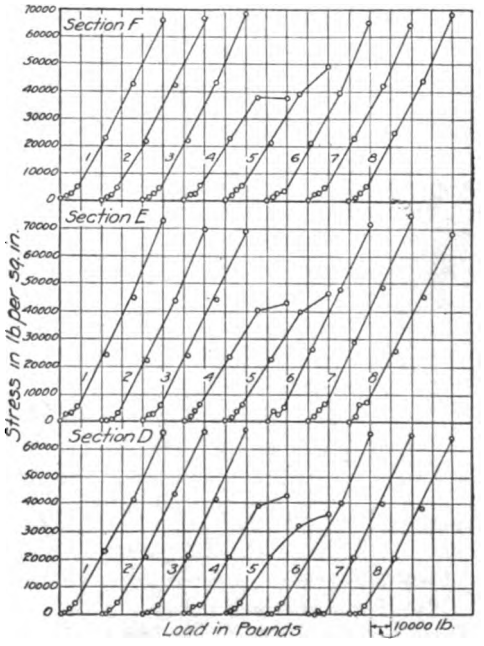

FIG. 44.—*Beam 8AAB1; one-half inch through and one-half inch lapped bars. Sections D–F*

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At a 22 000-pound load cracks opened up uniformly in all quarters and extended up into the beam about 7 inches. Between the load points tension cracks were quite uniformly spaced and extended into the beams about 6½ inches. No disturbance of the bond of the lapped bars could be seen at any of the observation points.

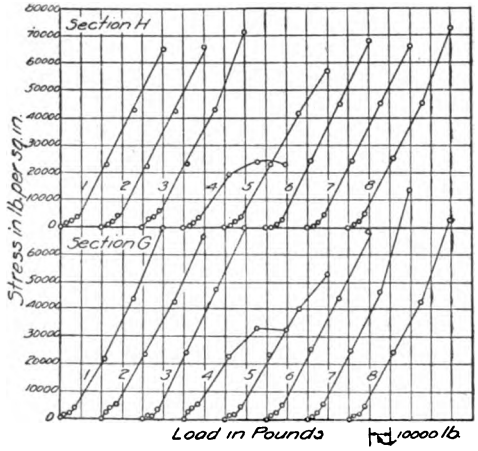

FIG. 45.—*Beam 8AAB1; one-half inch through and one-half inch lapped bars. Sections G–H*

At a load of 36 000 pounds all the cracks had lengthened an appreciable amount, but showed no great increase in width. Bar No. 4 showed the bond broken at the first two observation points on the unanchored end at readings K4 and I4. None of the other points showed any movement.

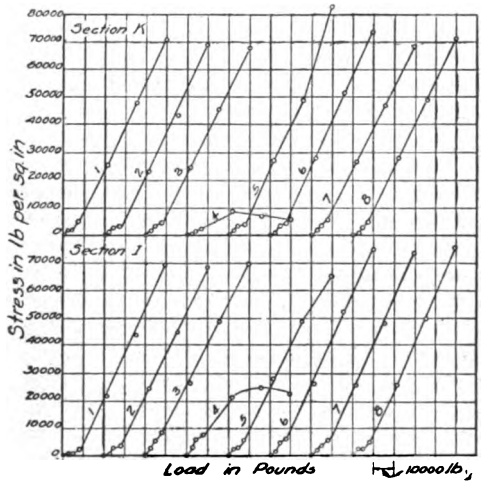

FIG. 46.—*Beam 8AAB1; one-half inch through and one-half inch lapped bars. Sections I–K*

At a 50 000-pound load the observation points on No. 4 lapped bar showed movement from the unanchored end to 4 inches beyond the center of the beam. On No. 5 bar the observation points from the unanchored end to the center of the beam showed movement. All of the cracks had increased in width, particularly the one

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at the east end of the lapped bar. The cracks outside of the load points had the direction of diagonal tension cracks.

The load increased to 55 150 pounds and then dropped slightly, and it was evident from the width of the tension cracks that the yield point of the steel had been reached. Strain gage readings were taken. The No. 4 bar showed slip from the unanchored end to 12 inches beyond the center of the beam. Bar No. 5 showed movement at all the observation points to the center of the beam. The load continued at about 54 000 pounds for some time, falling off very slowly with the machine running. The test was discontinued after the center deflection had reached 1.35 inches. The observation points on bar No. 5 from the unanchored end to 12 inches beyond the center of the beam and those on bar No. 4 from the unanchored end to 16 inches beyond the center of the beam showed movement. No slip of the through bars was observed.

(*c*) Beam 8AAC1, poured on November 29, was made ready for test December 27. This beam had six five-eighths-inch plain round through bars and two short sections of one-half-inch plain round bars lapped at the center of the beam for tension reinforcement. Strain gage readings were taken at loads of 1000, 3000, 5000, 10 000, 30 000, 50 000, and 51 000 pounds.

Further details: At a load of 3000 pounds one small crack developed in the southeast quarter at the end of the lapped bar.

At the 5000-pound load a similar crack showed in the northeast quarter.

At the 10 000-pound load a small crack in the southeast quarter extended about 3 inches parallel to the horizontal steel. Small tension cracks showed in all quarters between the load points and the supports.

At the load of 30 000 pounds several cracks appeared between the load points extending about halfway up the sides of the beam. Outside of the load points there were two or three cracks in each quarter, those in the northwest quarter having the direction of diagonal tension cracks. None of the observation points, consisting of patches of whitewash on the bars, showed disturbance of bond. The crack in the southeast quarter parallel to the horizontal bars showed a slight extension, and most of the cracks in all of the quarters showed branches parallel to the horizontal bars.

At a 50 000-pound load diagonal tension cracks showed in all the quarters. The cracks between the load points had opened wider and extended deeper into the beam. Bar No. 5 (lapped) showed slip at all the observation points from the free end to 8 inches beyond the center of the beam. Bar No. 4 showed movement at all the points from the free end to the center of the beam. The outside through bar on the north side showed slip at the observation point nearest the west load point. Horizontal cracks along the outside bars between load points became somewhat longer. Also outside of load points horizontal cracks extended toward the ends of the beams.

On starting the testing machine the load increased to 51 300 pounds, then dropped to 51 000 pounds. The size of the cracks showed that the yield point of the steel had been reached. A set of strain gage readings was taken. The first observation point at the unanchored end of bar No. 4 was the only one unbroken on the lapped bars. The whitewashed observation points were broken on the two outside bars at readings A7 and A8, also at K1, K2, K3, K5, and K8, showing the slip of these bars.

Loading was continued, and cracks opened up uniformly between the load points. The load continued at about 50 000 pounds for some time. After the center deflection had reached 0.87 inch the test was discontinued. The load at this point was 49 700 pounds.

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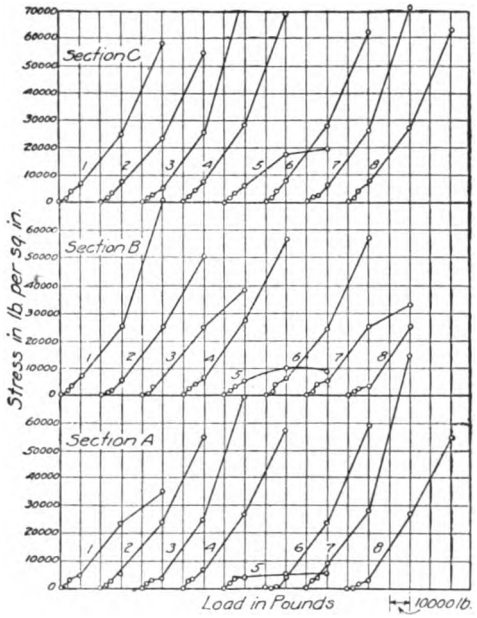

FIG. 47.—*Beam 8AAC1; five-eighths inch through and one-half inch lapped bars. Sections A-C*

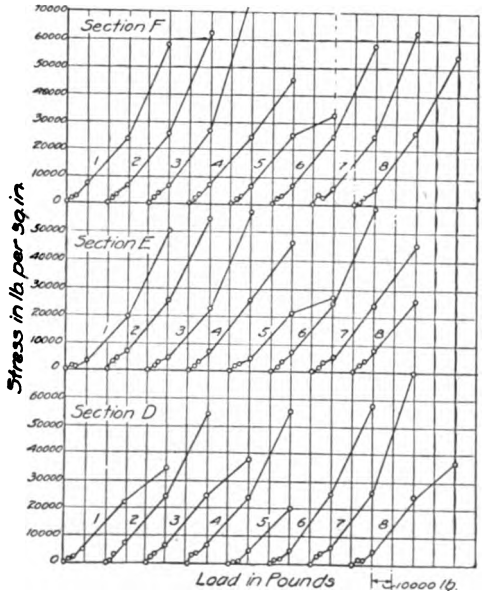

FIG. 48.—*Beam 8AAC1; five-eighths inch through and one-half inch lapped bars. Sections D-F*

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(*d*) Beam 8AAD1, poured on November 26, was set up for test December 24. This beam had for tension reinforcement six three-fourths-inch plain round through bars, anchored at both

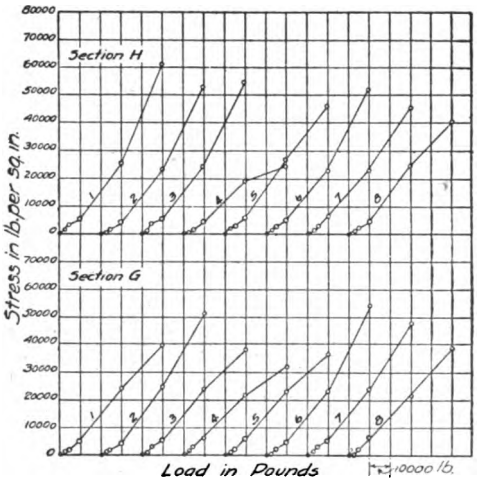

FIG. 49.—*Beam 8AAC1; five-eighths inch through and one-half inch lapped bars. Sections G–H*

ends of the beam, and two short sections of one-half-inch plain round bars lapped at the center of the beam. On removing the wooden blocks to drill the reinforcing bars for strain gage readings,

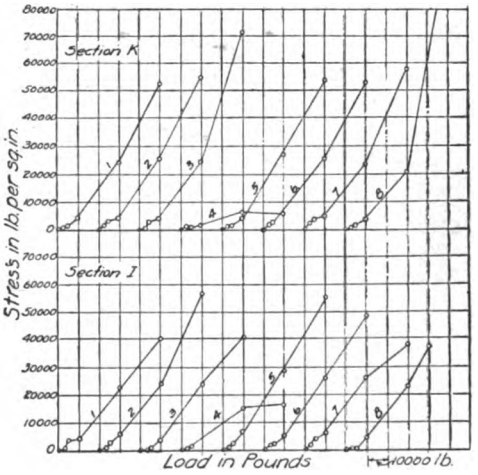

FIG. 50.—*Beam 8AAC1; five-eighths inch through and one-half inch lapped bars. Sections I–K*

the concrete around the bars was found to be porous. Fig. 51 shows a view of the bottom of this beam. All the concrete between the blocks was removed and a rich mortar carefully placed around the bars and rounded off above them. This was done a

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FIG. 51.—*Porous condition around reinforcing bars of beam 8AAD1*

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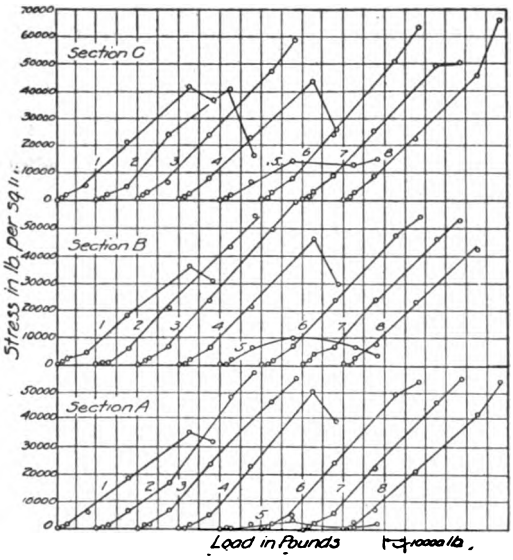

FIG. 52.—*Beam 8AAD1; three-fourths inch through and one-half inch lapped bars. Sections A–C*

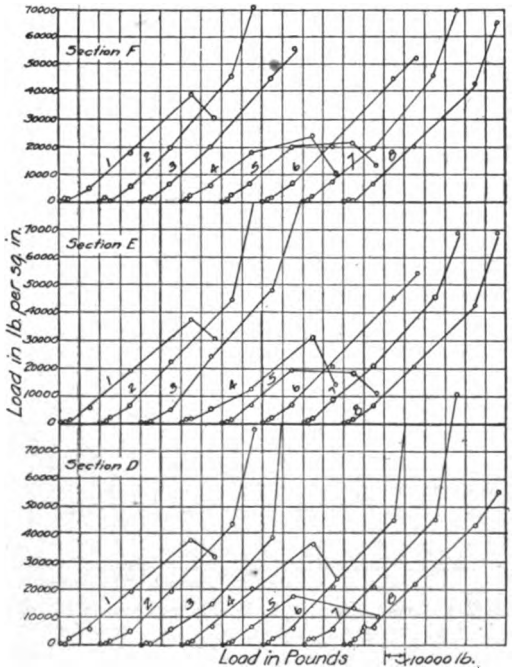

FIG. 53.—*Beam 8AAD1; three-fourths inch through and one-half inch lapped bars. Sections D–F*

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week before the test. Strain gage readings were taken at loads of 1000, 3000, 5000, 15 000, 35 000, 65 000, 76 400, and 73 000 pounds.

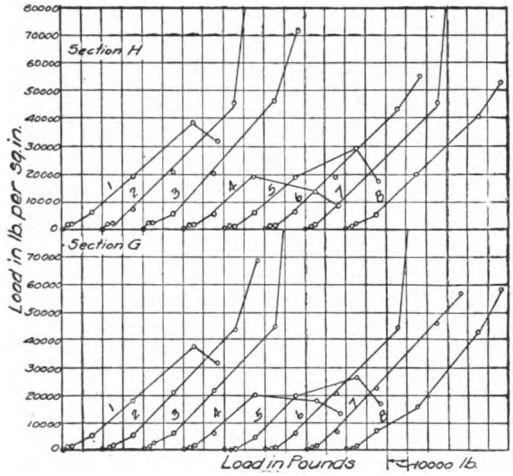

FIG. 54.—*Beam 8AAD1; three-fourths inch through and one-half inch lapped bars. Sections G–H.*

Further details: At a load of 3000 pounds fine cracks showed in the sections at the free ends of the lapped bars.

At a load of 5000 pounds more fine cracks showed, but except for one on the southeast face, none were over 2 inches long.

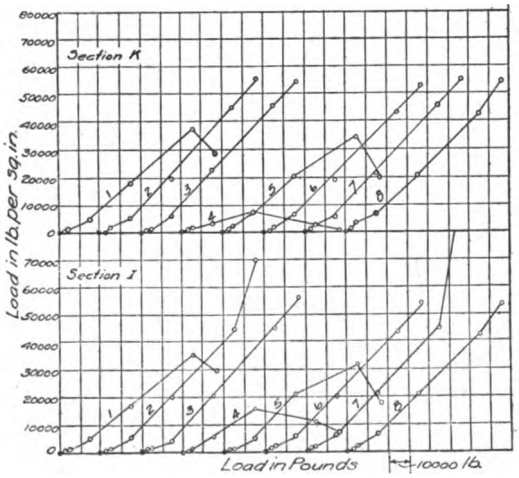

FIG. 55.—*Beam 8AAD1; three-fourths inch through and one-half inch lapped bars. Sections I–K*

At a load of 15 000 pounds cracks showed considerable development. Horizontal cracks appeared along the outside tension bars at each end of the beam. No breaking of the bond at the observation points was observed at this load.

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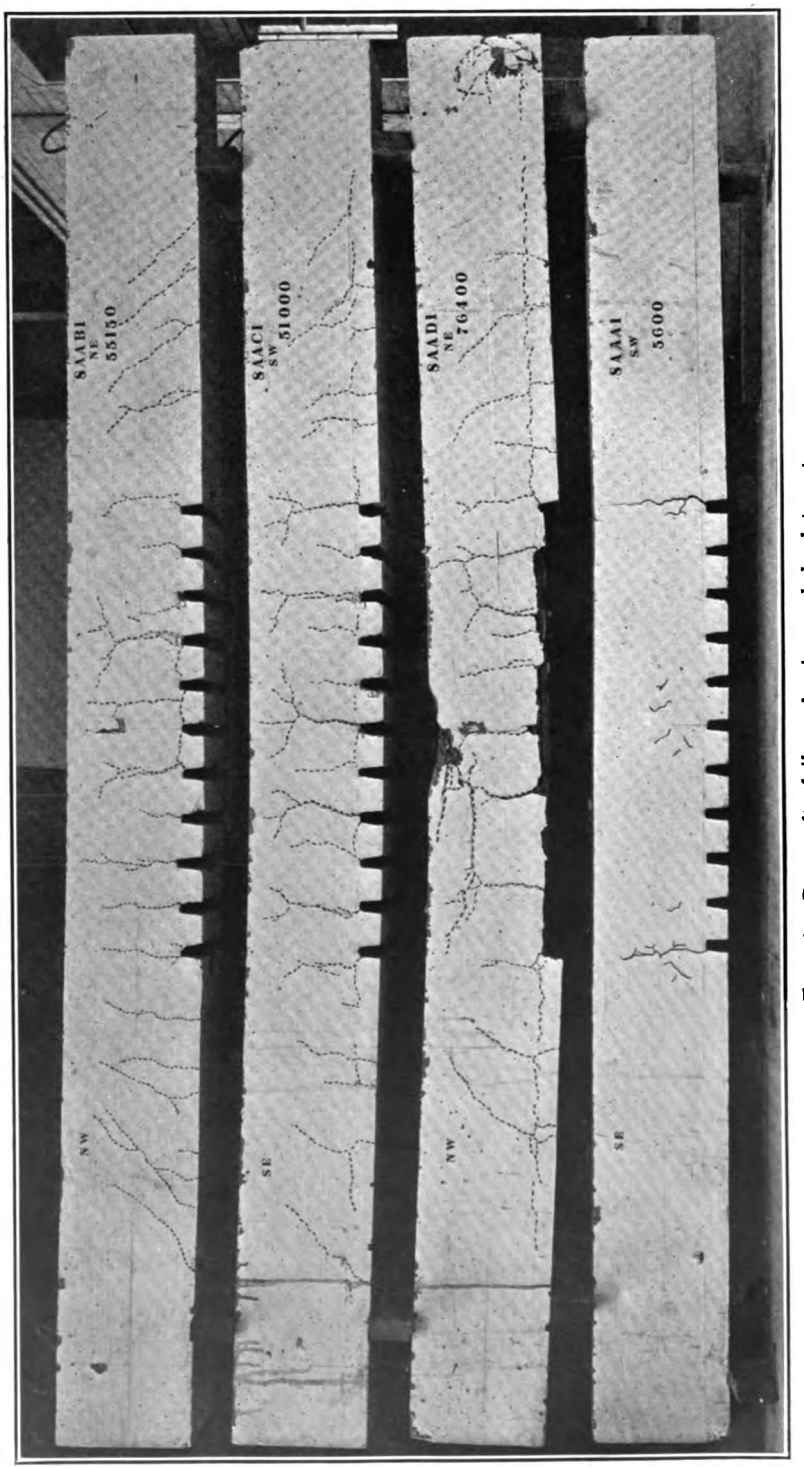

FIG. 56.—*Beams after failure, showing crack development*

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At a load of 35 000 pounds diagonal tension cracks showed in all quarters, and tension cracks appeared at the center of the beam. The first two observation points at the free end of each of the lapped bars showed slip.

With a load of 50 000 pounds on the beam, the whitewashed observation points showed movement of the lapped bars for four consecutive points from the free ends of the bars.

At a load of 65 000 pounds slip was observed at seven consecutive points. No movement of the through bars was observed. While standing at the load of 65 000 pounds, two more observation points on bar No. 5 and one more on bar No. 4 showed movement.

At a load of 76 400 pounds cracks following the outer horizontal bars showed throughout the full length of the bars, following around the hooks at the southwest and northeast corners. The whitewashed observation points showed that the two outer through bars had slipped. One of the lapped bars showed slip at all the points of observation. The other bar showed slip at all but the observation point nearest the free end. The concrete started to split off at the southwest corner around the hooked end of the outer through bar. There was originally a three-thirty-seconds-inch covering of concrete over the bar at this point. This reading was taken at the maximum load. Although the shearing stress at this load was 465 pounds per square inch there was no indication that failure due to this cause was approaching.

The load dropped off to 73 000 pounds and remained at this point for some time with the machine running. The size of cracks indicated that the stress in the steel was reaching the yield point, and so a set of readings was taken. The readings at this load showed the steel to be overstressed in the locality of these cracks.

Loading was resumed at constant speed. The load fell off to 68 000 pounds and continued at this point for some time, or until a failure by compression of the concrete on the top of the beam at the center of the span occurred.

This failure was sudden and accompanied by a sharp report. On the northeast corner the concrete had spalled off in about the same manner as at the southwest corner, due to slipping of the hooked end of the outside bar. The other corners were not affected. The other three-fourths-inch bars did not show any failure of bond except at the region where the stress exceeded the yield point of the steel. All the points of observation on the lapped bars which were covered with whitewash showed movement. The movement of the ends of these bars was over one-half inch.

7. GENERAL RESULTS

Table 14 gives some of the results of these tests.

Fig. 56 shows a view of all the beams after test. The cracks are shown by broken lines. Fig. 63 gives the load-deflection diagrams for all the beams. Figs. 43 to 56 give load-stress diagrams for all the gage lines for various sections of the portions of the beams under uniform bending moment.

8. DISTRIBUTION OF TENSILE STRESS AMONG BARS

Figs. 57 to 62 give the intensity of tensile stress among the bars across the beams at various sections. Two loads are represented in each case. The average tensile stress for all bars at each load is shown by a broken line. A study of these diagrams shows that there was a point on each lapped bar beyond which there was a falling off of the stress due to the slipping of the free end of the bar. Proceeding from this point toward the end of the bar, the intensity of stress steadily decreased, and the stress in the other

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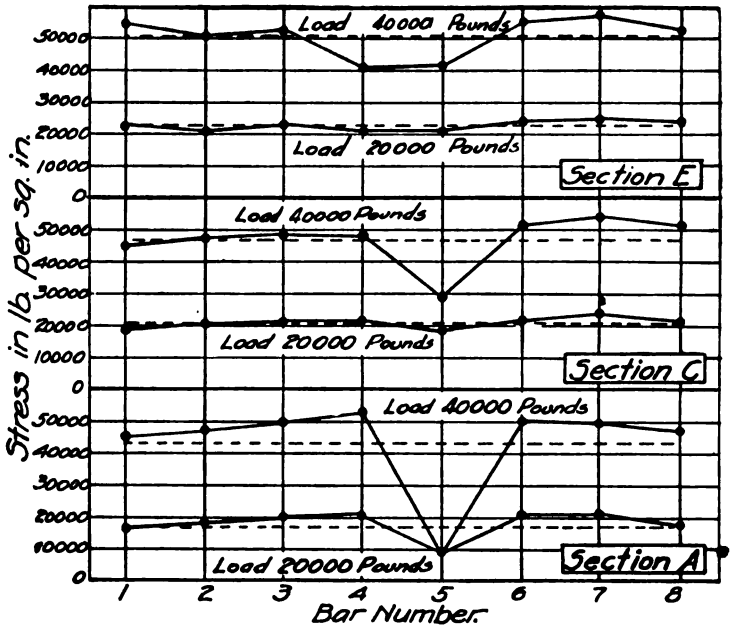

FIG. 57.—*Beam 8AABI. Sections A, C, and E*

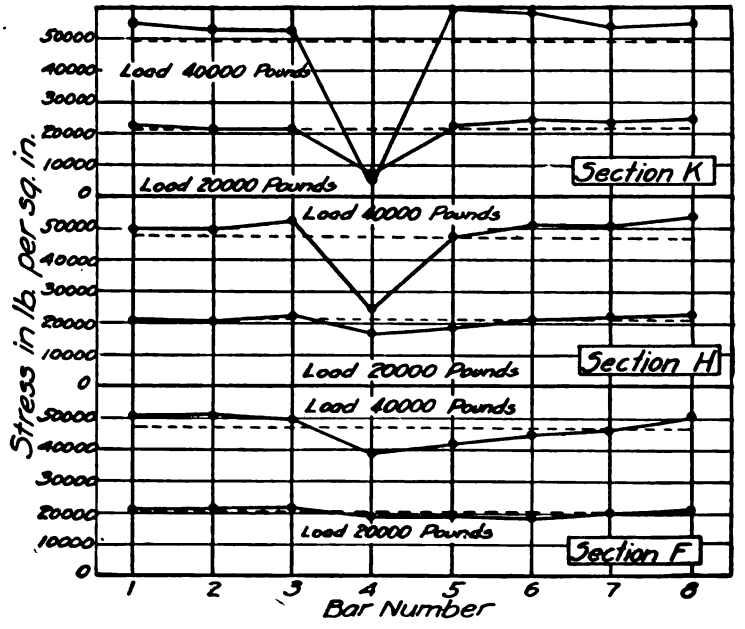

FIG. 58.—*Beam 8AABI. Sections F, H, and K*

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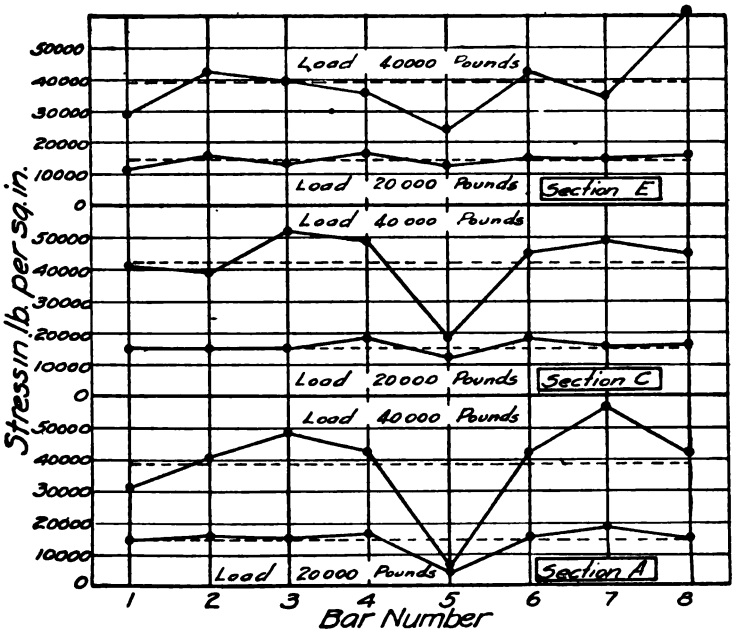

FIG. 59.—*Beam 8AAC1. Sections A, C, and E*

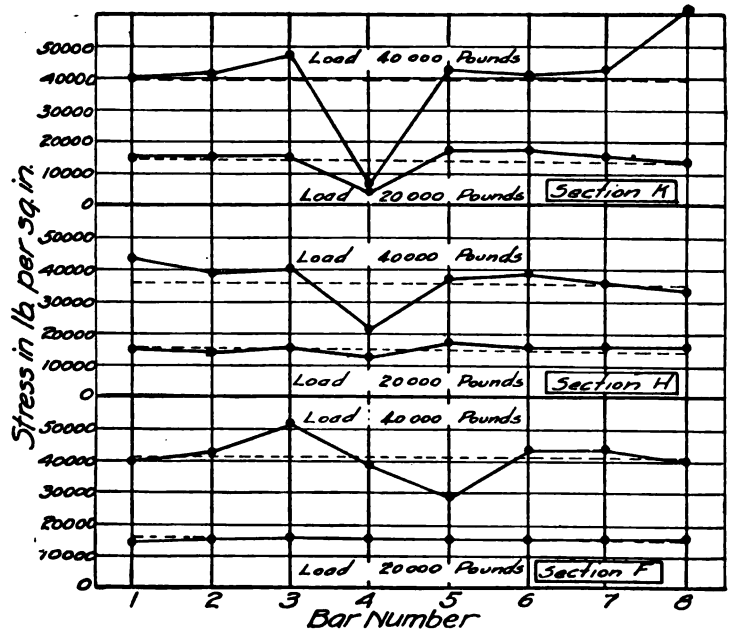

FIG. 60.—*Beam 8AAC1. Sections F, H, and K*

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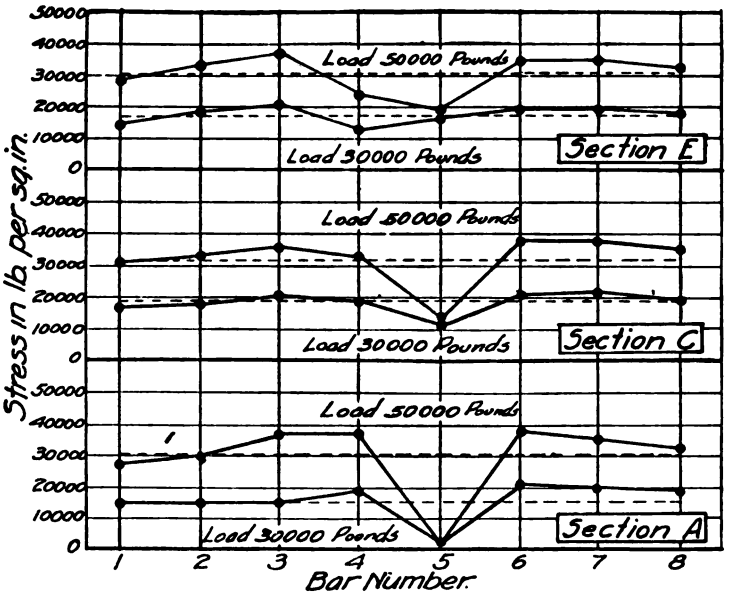

FIG. 61.—*Beam 8AAD1. Sections A, C, and E*

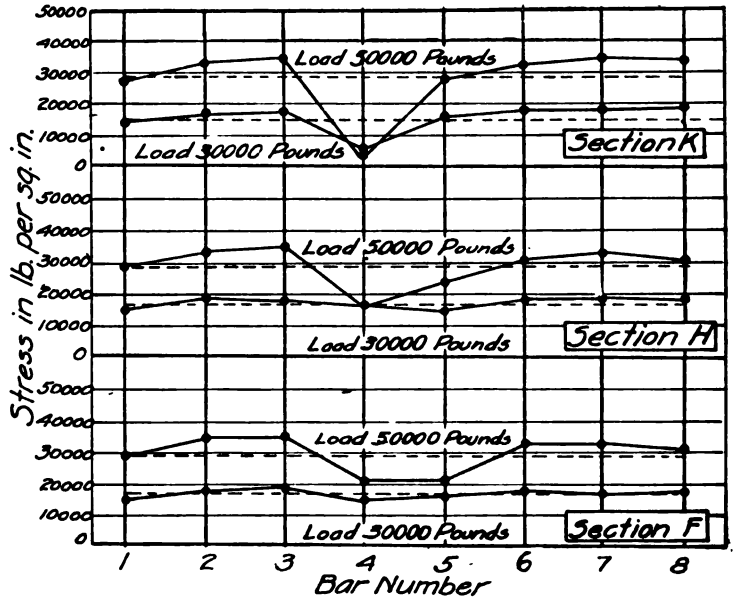

FIG. 62.—*Beam 8AAD1. Sections F, H, and K*

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bars of the beams increased correspondingly at the same sections. In most cases the increase in stress was greater in the through bars immediately adjacent to the lapped bars than in the other bars of the beam. However, this tendency is not pronounced, and there is no evidence that it is affected by the variation in the proportional areas of through and of lapped bars.

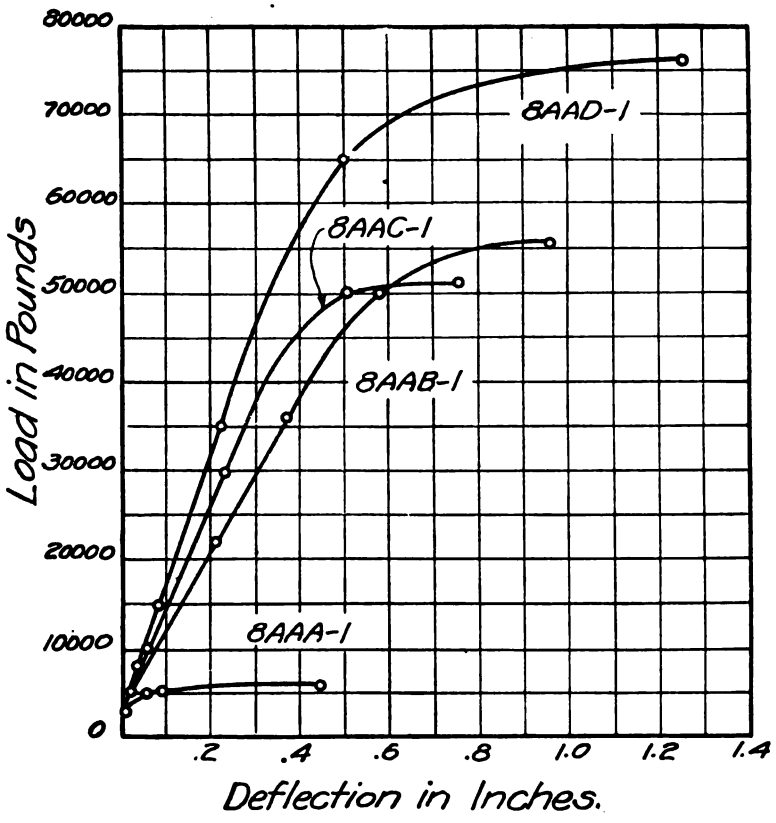

FIG. 63.—*Load-deflection diagrams for all beams*

9. DISTRIBUTION OF TENSILE STRESS ALONG BARS

Figs. 64 and 65 show the distribution of tensile stress as measured on the lapped bars for each of the beams. For the beams with through bars, the average tensile stress for all the through bars at the corresponding sections is shown by a broken line. Curves are given for loads giving computed stresses of 16 000 and 40 000 pounds per square inch in the tension bars.

The curve for beam 8AAD1, Fig. 65, at a computed stress of 40 000 pounds per square inch, shows that the stress in one of the bars of the lap did not become equal to that in the adjacent through bars within the length of the splice. This was probably

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due to the porous condition of the original concrete around the tension reinforcement of this beam.

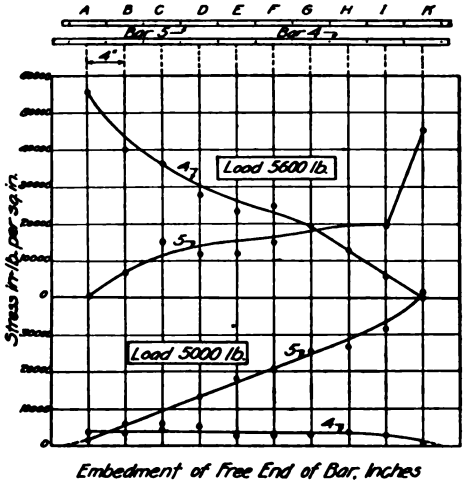

FIG. 64.—*Distribution of tensile stress along lapped bar in beam 8AAA1*

The curves for beam 8AAA1, Fig. 64, are for a higher computed stress than 40 000 pounds per square inch. At the load which gives 40 000 pounds per square inch computed stress, cracking had

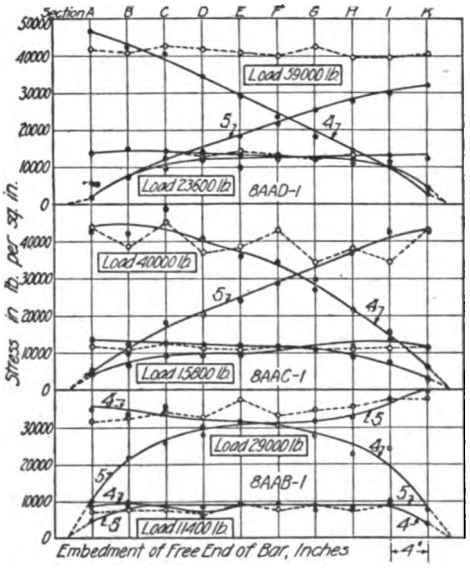

FIG. 65.—*Distribution of tensile stress along lapped bar in beams 8AAB1, 8AAC1, and 8AAD1*

not developed sufficiently to cause the tension reinforcement to carry all the tensile stress unaided by the concrete.

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10. DISTRIBUTION OF BOND STRESS ALONG BARS

Figs. 66 and 67 show the distribution of bond stress along the bars of the lap for the three beams with through bars. Each curve is the average for the two lapped bars of a beam. The values plotted are proportional to the differences in tensile stress as measured by the strain gage on consecutive gage lines, starting at the free end of the bar. Curves for two loads are given. The curve for the lower load of each beam shows the distribution of bond stress along the bar when the bond stress at the free end of the bar is at or near the maximum. The curves for the higher loads

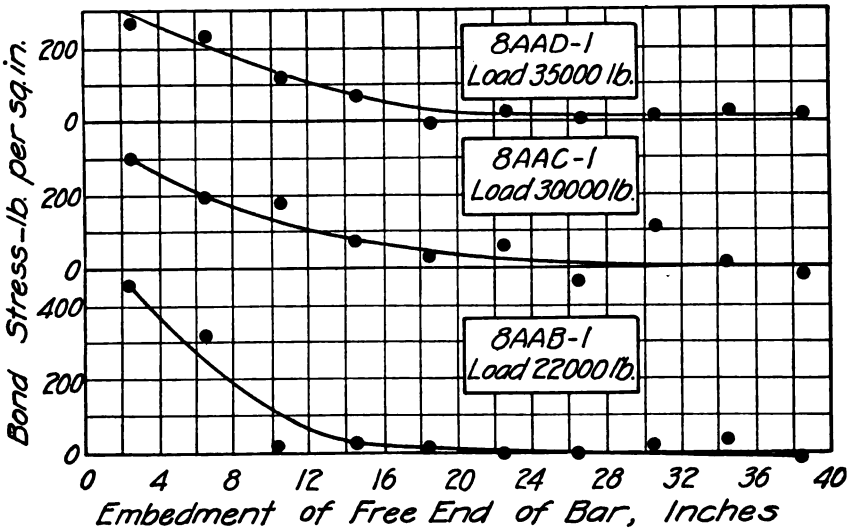

FIG. 66.—*Distribution of bond stress along lapped bars in beams 8AAB1, 8AAC1, and 8AAD1*

have the same general form, but here the bond stress does not decrease to zero before the end of the lap of the adjacent bar is reached.

In the recorded bond stresses no correction has been made for the area of steel left exposed by the wooden blocks placed in the concrete for the purpose of locating the strain gage readings on the steel. This area would amount to about 5 per cent of the total surface area of the steel between gage points. Near the unanchored end of the lapped bars individual maximum bond stress values may be found which are about 75 per cent of the average values given in the first part of this report for pull-out tests of uncoated plain square bars.

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11. SLIP OF BARS

Fig. 68 shows the progress of slip along the bars of the lap from the free ends to the ends of the splice for the same loads as those for which the bond-stress curves are given. Each curve is the average for the two lapped bars of a beam.

The slip for any point on the lapped bar at any load was obtained by subtracting the sum of the measured deformations between that point and a point where evidently no slip had occurred from the sum of the corresponding deformations of the adjacent through bars. The original plan of measuring the slip of the end of the lapped bar was not used on account of the fact that in all cases a crack opened between the stub and the end of the bar. This caused an indeterminate error in the measurement of the slip, as taken by the strain gage, equal to the width of the crack.

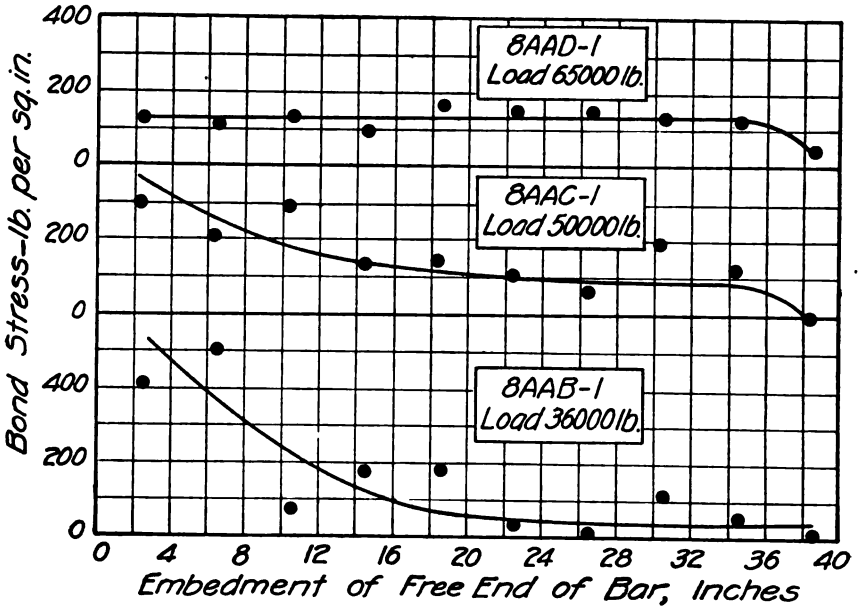

FIG. 67.—*Distribution of bond stress along lapped bars in beams 8AAB1, 8AAC1, and 8AAD1 at higher loads*

A comparison of the bond-stress curves for the load which gives the maximum bond stress at the end of the lapped bar, Fig. 66, with the slip curves for the same loads, Fig. 68, shows that the maximum bond stress at the end of the bar occurred at a very small slip. However, the bond stress at the end of the bar fell off slowly from the maximum, and with a slip twice as great

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as that which gave the maximum the bond stress was not very much reduced. This action is evidently due to frictional bond resistance. The slipping of the hooked bars of beam 8AAD1

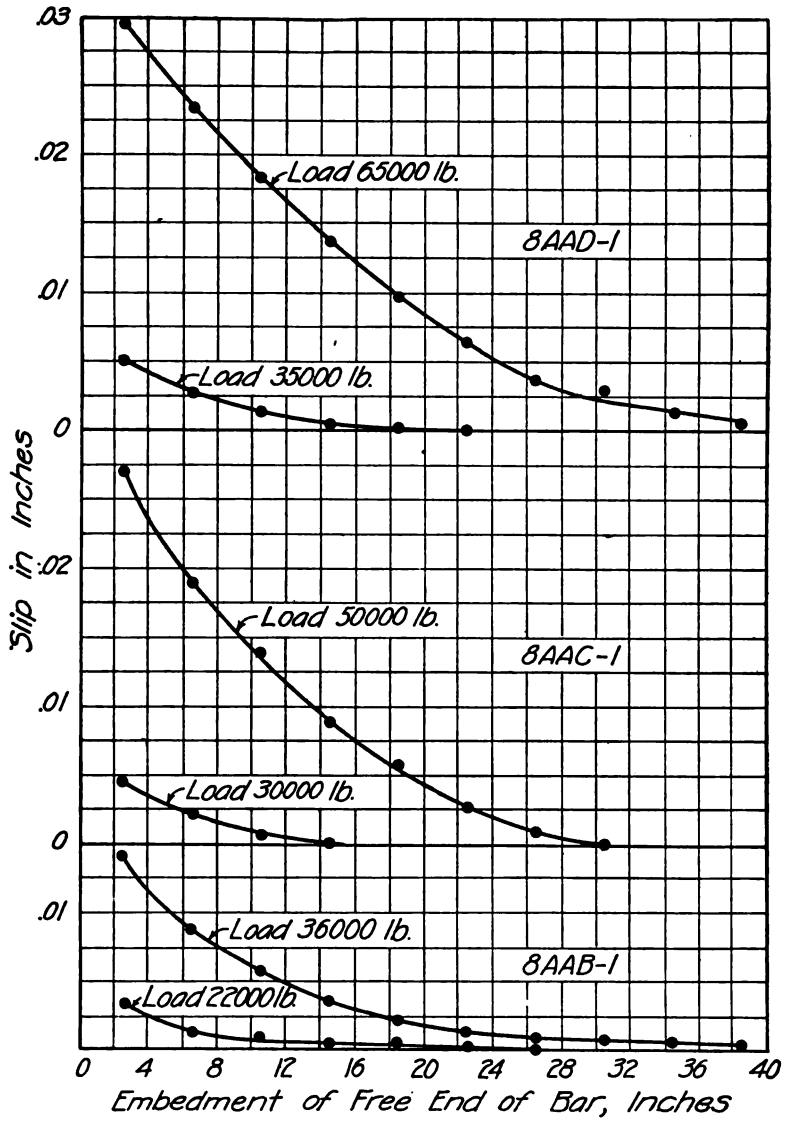

FIG. 68.—*Distribution of slip along lapped bars in beams 8AAB1, 8AAC1, and 8AAD1*

at the maximum load indicates that the bond resistance was insufficient to develop the full tension which existed in the bar and that the hooks were coming into action in furnishing anchorage. The computed bond stress at the maximum load was about

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300 pounds per square inch or 6 per cent of the compressive strength of the concrete and the computed tensile stress was about 62 000 pounds per square inch. The lack of concrete in contact with the bars over a considerable distance probably contributed to this slipping. The condition of this beam is shown in Fig. 51. The same figure indicates that at least in the middle portion of the beam the outside bars were much closer to the sides of the beam than is called for in Fig. 39. If the condition at the end of the beam was similar to that at the center of the span, the bars were closer to the side of the beam than ordinary good practice would approve and this defect is probably responsible for the bursting of the hooked end of the bar through the side of the beam.

12. LENGTH OF LAP REQUIRED FOR BEAMS WITH THROUGH BARS

To serve as a basis for analysis, it may be assumed that for any given tensile stress the safe length of lap is the distance from the free end of the bar to the point where the tensile stress in the lapped bar becomes constant or equal to the stress in an adjacent through bar. If the lap furnished is less than this, the tensile stresses in the through bars at the ends of the lap must be greater than the value computed by the equation

$$f_s = M/A_s jd,$$

unless the difference in stress is taken up by the concrete in tension. This is based on the assumption that the entire lap lies within a region of constant bending moment.

With this assumption as a basis, Fig. 69 has been prepared. The data from the beam without through bars were not used in this figure. The amount of reinforcement for this beam was very small; consequently, the concrete took a large portion of the tensile stress, and the stress curves for the steel were too erratic for use in this way. The ordinates of the points given represent the distance from the ends of the lapped bars at which the measured stresses shown as abscissas were developed. A straight line represents fairly well the mean of the values shown by the points. The equation of this line is

$s = 0.00059\ f_s$, where (1)

s = distance in inches from end of lapped bar to a point where the bond stress becomes zero under a given load, and

f_s = the tensile unit stress computed from the corresponding bending moment.

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It is possible that further investigation, using plain bars of diameters other than one-half inch, or deformed bars, would yield a different constant in this equation. A further modification would probably be produced by using concrete mixed in other proportions than 1 : 1 : 1.

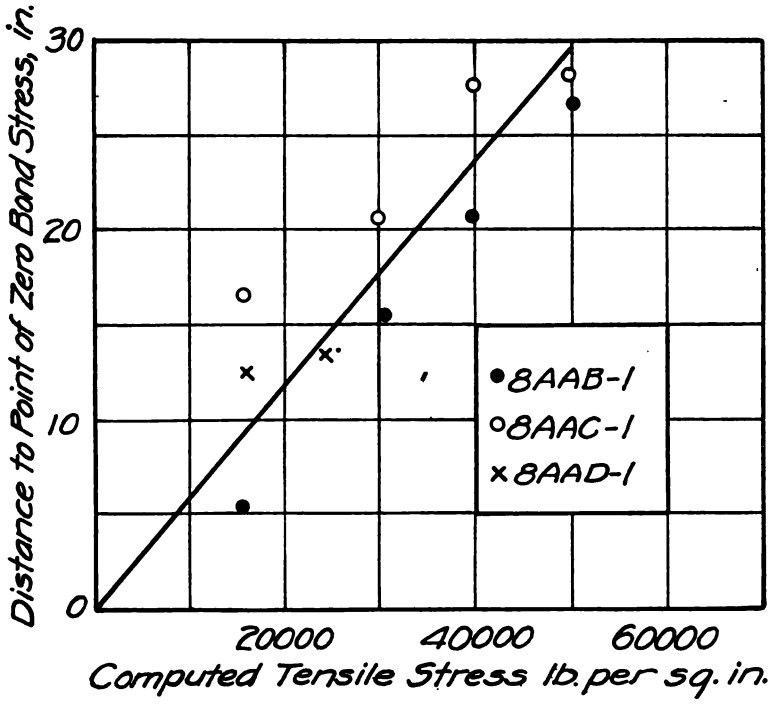

FIG. 69.—*Relation of tensile stress to length of lap required*

It is apparent that the total tensile stress in a lapped bar at the section where the bond stress becomes zero is

$$\frac{\pi a^2 f_s}{4} = \pi a u s, \text{ where} \qquad (2)$$

a = diameter of bar, and
u = average bond stress in distance s.

From this equation,

$$f_s = \frac{4\ us}{a}. \qquad (3)$$

Substituting in equation (1)

$$s = 0.00059\ \frac{4\ us}{a}, \text{ and} \qquad (4)$$

$$u = \frac{a}{4 \times 0.00059}. \qquad (5)$$

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For lapped bars one-half inch in diameter the value of u from this equation equals 210 pounds per square inch. Equation (5) indicates that the value of the bond stress was independent of the tensile stress developed. The acceptance of this form of equation is equivalent to concluding that the average bond stress over the length s was the same at all stages of the test, but that the length s increased as the tensile stress increased.

Reference to beam 8AAB1, Figs. 66 and 67, shows that the distance s from the end of the bar at which the bond stress approximates zero was greater for the load of 36 000 pounds than for the load of 22 000 pounds, but that the average bond stress was not far from 210 pounds per square inch in both cases. For beams 8AAC1 and 8AAD1 the average bond stress was less than this. Apparently the conclusion that the average bond stress developed over the length of lap is independent of the tensile stress developed in the bars is not fully justified, but as a basis for design this assumption is probably exact enough.

It is generally assumed that the length of embedment required to develop the tensile strength of a bar is directly proportional to the diameter of the bar. In this investigation it was not feasible to test a sufficient number of beams with lapped bars to confirm this assumption, but it is here accepted as logically to be expected. Since these tests were made on one-half-inch bars, the equation becomes by substitution from equation (1),

$$\frac{s}{a}=\frac{0.00059}{1/2}f_s=0.00118\,f_s, \tag{6}$$

or,

$$s=0.00118\,f_s a. \tag{7}$$

The stress in the lapped bar just outside the lap is the f_s of this equation. If this is less than the average stress at this section calculated by the equation

$$f_s=\frac{M}{A_s\,jd},$$

then the stress in the through bars must be greater unless the difference is taken up by the tensile resistance of the concrete. Consequently, in using equation (7) to compute the minimum safe length of lap, the value of f_s should be the yield point of the steel. If f_s be taken less than this, the yield point will be passed in the through bars before it is reached in the lapped bars. Assuming the yield point to be 40 000 pounds per square inch, and that the concrete has the same strength as the concrete used in these tests,

$$s=47.2a. \tag{8}$$

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Under a working load the total tensile stress developed in the lapped bar within the distance s from its free end is as stated in equation (2). In this case the values of u and s to be used are the working stress. From this equation, using $f_s = 16\,000$,

$$u = \frac{af_s}{4s} = 212 \text{ pounds per square inch.}$$

From Table 14 the average compressive strength of the control cylinders was 5020 pounds per square inch. Assuming that the bond resistance is proportional to the compressive strength of the concrete,[9] f'_c,

$$\frac{u}{f'_c} = \frac{85}{5020} = 0.0169, \text{ and}$$

$$u = 0.0169\, f'_c \qquad (9)$$

13. EFFECT OF VARIATION OF AREA OF THROUGH BARS ON LENGTH OF LAP REQUIRED

A study of Fig. 69 indicates that within the limits of the data used for this diagram a variation in the ratio of the area of the through bars to that of the lapped bars had little if any effect on the length of lap required. However, in these tests the least area of through bars was six times that of the lapped bars. With smaller proportions of through bars it is possible that a greater length of lap might have been needed. An extreme case is represented by beam 8AAA1, which had no through bars. This beam failed by the pulling out of one of the bars, although the lap was 80 diameters. This occurred at a load which gave a computed tensile stress in the steel of 50 000 pounds per square inch, and a measured stress of 55 000 pounds per square inch at one point. While this stress was high, it was less than the yield point of the steel. The evidence afforded by the failure of beam 8AAA1 indicates that a greater length of lap is required for beams in which all the bars are lapped than for beams in which there are through bars. Further investigation along this line is desirable.

14. SUMMARY OF RESULTS

The following statements summarize the results of the beam tests reported above:

(*a*) Proceeding along a lapped bar toward its unanchored end from the point where the stress in it begins to diminish, the stress lost is picked up by the other bars of the beam. In a majority of

[9] Abrams, Tests of Bond Between Steel and Concrete, Bull. 71, Eng. Exp. Sta., Univ. of Illinois, p. 94, 1913.

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cases a larger amount of the stress lost by the lapped bar was picked up by bars immediately adjacent than by the other bars of the beam.

(*b*) The maximum bond stress developed by the lapped bars was about 75 per cent of that reported for pull-out tests of uncoated plain square bars in Division II of this paper. The maximum bond stress occurred at a smaller amount of slip of bar for the beams than for the pull-out tests.

(*c*) The average bond stress in the lapped bars was practically independent of the tensile stress in the bars, but the length over which it was effective varied with the magnitude of the stress.

(*d*) For the purpose of design the minimum safe length of lap may be taken as the distance from the unanchored end of the lapped bar to a point on the bar where the bond stress is zero, when the tensile stress in the steel is at the yield point. With this assumption the tests indicate that using steel which has a yield point stress of 40 000 pounds per square inch, and with continuous bars adjacent to the lapped bars and with concrete of the grade here used, the lap should be about 48 diameters. Care should be used in applying this relation under other conditions.

(*e*) The tests of beams with through bars do not show that differences in the ratio of the area of the through bars to the total area cause any variation in the length of lap required. The test of the beam with no through bars indicated that when all the bars are lapped, a longer lap is needed than when through bars are present. Further investigation is needed on this phase of the subject.

(*f*) A shearing stress of 465 pounds per square inch was developed in one of the beams without sign of approaching failure by diagonal tension.

IV. TESTS ON STIRRUP PULL-OUT SPECIMENS

1. PRELIMINARY

The tests described in the following pages of this paper were made on 12 test specimens designed to furnish useful information on the strength of stirrups. The dimensions of the specimens and reinforcement used conformed to a typical section through the shell and frame in a concrete oil-tank ship which was built by the Emergency Fleet Corporation. In such a section, due to greater stiffness in the frame than in the shell, the hydrostatic pressure of the cargo tends to force the shell away from the frame, thus

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adding direct tension to the tension induced in the stirrups by shearing stresses in the frame. Under such circumstances the efficiency of the anchorage of the stirrups is of great importance.

2. DESCRIPTION OF SPECIMENS

The specimen consisted of a concrete slab 4 inches thick, cast monolithic, with a concrete rib 6½ inches wide and 12 inches deep. The slab was 1 foot 10½ inches long and 1 foot 4 inches wide. The rib was 2 feet 4 inches long and was attached to the slab along its center line in the short direction; it projected 6

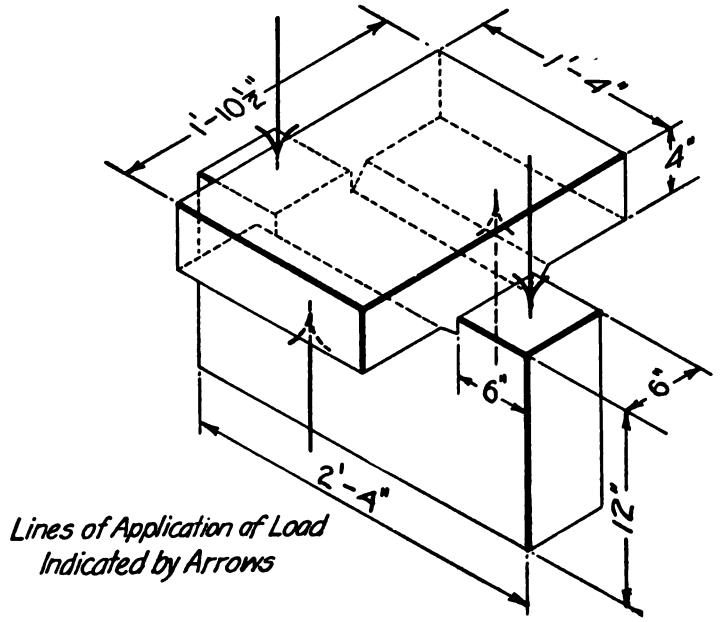

FIG. 70.—*Isometric view of specimen used in stirrup pull-out tests*

inches beyond the slab at each end to afford a bearing for the downward load. Fig. 70 is an isometric view of the test specimen.

The reinforcement in the prism and in the slab was placed so as to make the specimen as much like the corresponding section of the ship as possible. The three bars placed at the junction of the prism and the slab made the section weak in resistance to a tensile stress tending to separate the slab from the rib. The specimens were designated by letters from A to L, inclusive.

Three types of stirrups were used, as shown in Fig. 71, and designated as Types I, II, and III, respectively. The stirrups were made from one-half-inch plain round steel bars, bent into U shape with hooked ends. Type I stirrups had the hooked ends

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bent perpendicular to the plane of the legs and were used in specimens D, E, F, K, and L. Type II stirrups had the ends bent into loops through an angle of 270°, in which the planes of the loops were perpendicular to the plane of the legs. Stirrups of Type II were used in specimens A, B, C, G, H, and I. Type III stirrups had the ends bent outward at 90° from the legs and in the same plane. This type was used in specimen J.

Two stirrups of the same type were used in each specimen. They were placed 6 inches apart. The hooked ends of the stirrups were arranged at different depths in the slab member of the specimens and, in consequence, varied in relation to the position of the slab reinforcement. The distance between the hooks and lower surface of the slab for each specimen is shown in Table 15. The loops in the stirrups used in specimens G, H, and I were

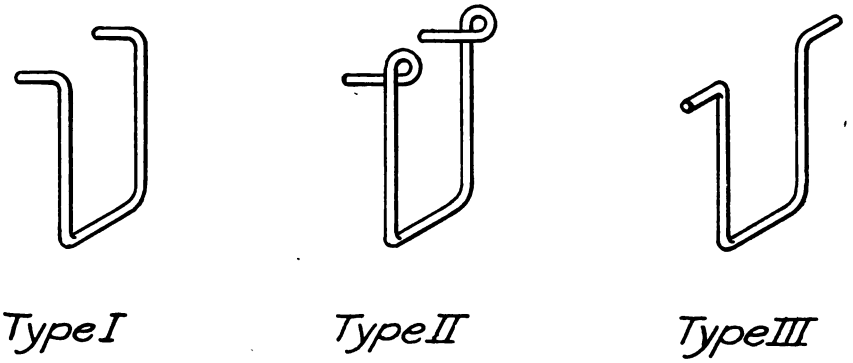

FIG. 71.—*Types of stirrups*

stuffed with mineral wool in order to exclude concrete during the pouring and to produce the condition of a void within the loop, a condition which is sometimes found in actual construction.

Fig. 72 shows the elevation and cross section of a typical specimen with the reinforcement indicated.

3. MAKING OF SPECIMENS

The concrete was mixed in the proportions 1:⅔:1⅓, measured by volume. Lehigh Portland cement and burnt-clay aggregates manufactured by the Atlas Portland Cement Co. were used. The consistency of the concrete was determined for each batch by the slump test; the average drop in a 6 by 12 inch cylinder was 9 inches when the mold was lifted. The concrete was representative of that used in the construction of concrete ships. In specimens A to F, inclusive, it was necessary to substitute sand and gravel

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in the concrete used to a depth of about 6 inches in the rib of the specimen. This change was required on account of a shortage in the amount of burnt-clay aggregates available. The results of the tests show no effect that may be attributed to this change in mix. The specimens were poured from the top of the slab after the reinforcement was wired in position.

4. AUXILIARY SPECIMENS

To determine the strength of the concrete at the same ages as the stirrup pull-out specimens when they were tested, 19 control cylinders, 3 by 6 inches in size, were molded from the batches mixed for the pull-out specimens. The average results of the

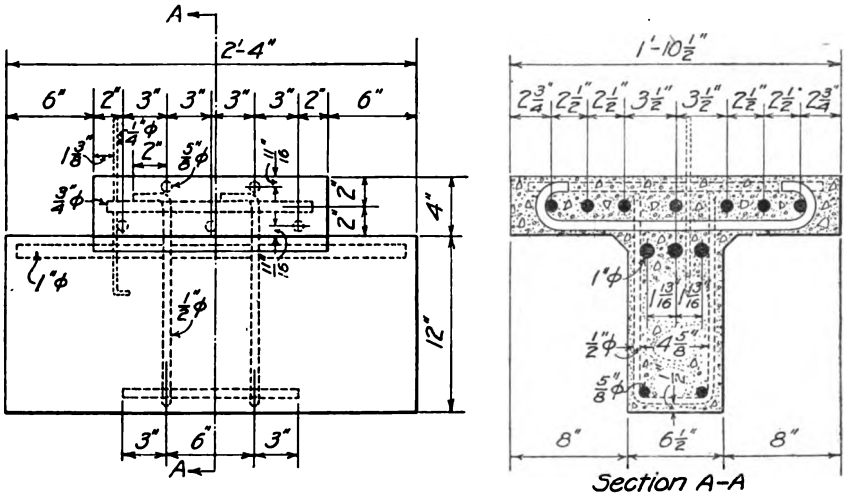

FIG. 72.—*Details of stirrup pull-out specimen*

compression tests on these cylinders are given in Table 15 and show a range of strength from 3900 to 4900 pounds per square inch.

5. METHOD OF TESTING

The stirrup pull-out specimens were tested in an Olsen testing machine of 200 000 pounds capacity. Fig. 73 shows a specimen arranged for testing.

A one-fourth inch plain round bar extended vertically through the slab of each specimen and 4 inches down into the rib. The bar was located 1⅜ inches from one edge of the slab. The lower end was anchored in the rib by means of a short L hook; the other end projected 4 inches above the slab. The upper half of the embedded length, or that passing through the slab member,

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was coated with cup grease to destroy the bond resistance between the concrete and the rod. The purpose of this rod was to hold an Ames dial gage in contact with the slab during the test in order to determine, if possible, the instant when separation between the slab and the rib started.

The manner of loading the specimen was such as to pull the slab away from the rib. The overhanging ends of the slab member were supported on bearing plates and steel rollers by I beams, which were laid on the weighing table of the testing machine. The load was applied by the downward movement of the pulling head of the machine acting through a steel beam and spherical

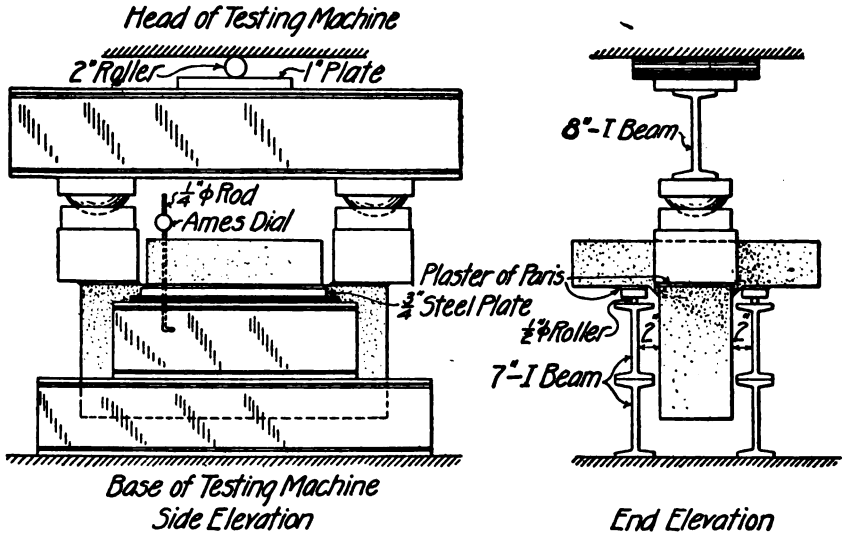

FIG. 73.—*Arrangement for testing stirrup pull-out specimen*

bearing blocks on the ends of the rib member of the specimen. The machine was stopped for examination of the specimen after each 3000-pound increment of load was applied.

The first indication of impending failure came by the sudden appearance of cracks in the plane at the junction of the slab and the rib of the specimen. Use of the Ames gage was abandoned after testing two or three specimens, as it gave no indications of value prior to failure.

6. RESULTS OF TESTS

Table 15 shows that the first crack occurred at about the same total load in all of the specimens except B and C, in which the load was somewhat higher. The sketches in Table 15 show that

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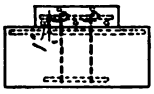
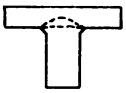
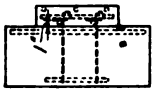
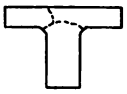
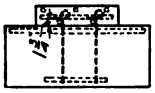
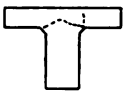
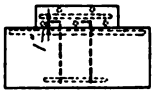
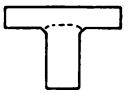
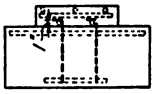
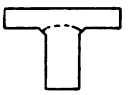
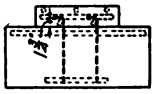
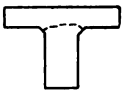
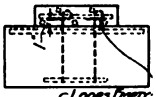
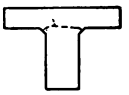
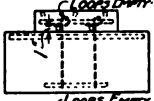
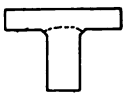
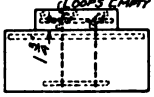
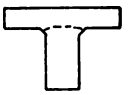
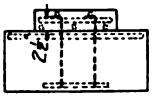
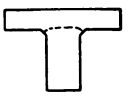
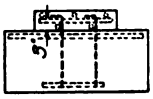
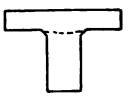
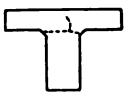
TABLE 15.—Tests of Stirrup Pull-Out Specimens

Specimen and type	Age	Compressive strength of concrete	Load at first crack	Computed tensile stress in stirrups at first crack	Maximum load	Computed tensile stress in stirrups at maximum load	View showing position of stirrups	End view showing first crack
	Days.	Lbs./in.²	Pounds.	Lbs./in.²	Pounds.	Lbs./in.²		
A-II	31	4000	14 000	17 800	22 600	28 800		
B-II	36	4200	19 000	24 200	23 500	29 900		
C-II	31	4000	18 000	22 900	32 700	41 600		
D-I	32	4100	14 000	17 800	15 300	19 500		
E-I	31	4000	12 500	15 900	12 500	15 900		
F-I	30	3900	15 000	19 100	19 500	24 800		
G-II	17	4900	15 000	19 100	21 700	27 600		
H-II	18	4900	13 500	17 200	19 000	24 200		
I-II	12	4500	15 000	19 100	18 500	23 600		
J-I	14	4600	16 500	21 000	25 000	31 800		
K-I	17	4900	14 000	17 800	16 000	20 400		
L-III	13	4600	12 000	15 300	19 000	24 200		

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for most of the specimens the surface of rupture was substantially a plane coinciding with the lower surface of the slab. They indicate that for specimens B and C the surface of rupture had an area somewhat greater than that for the other specimens. If this could be taken into account in the calculations, the tensile stress at the time of the formation of the first crack might be found to be nearly the same for all specimens. Immediately after the failure of the concrete the entire load on the specimen was transferred to the stirrups. The position of the surface of rupture caused by the first crack had an important bearing upon the total resistance of the stirrup anchorage, since in many cases it passed so close to the hooks as to leave them little covering of concrete.

The tests show that Type II stirrups were much more effective when the loops were filled with concrete than when they were empty, and under such conditions were stronger than either Type I or Type III. Even when the loops were not filled with concrete the stirrups of Type II were better on the average than those of Type I. The one specimen having Type III stirrups gave greater load at the first crack and greater maximum load than any of the specimens having stirrups of Type I. The first crack probably had less effect on the embedment of the hooks in the Type III stirrups than in the stirrups of Type I.

In general, the stirrups which had the deepest embedment of the hooked and looped ends showed the greatest strength. An embedment of not less than 2 inches is apparently desirable.

Comparison of specimen D with E and G with H indicates that no increase in effectiveness was obtained by having the stirrup hooks rest on a bar of steel in the slab reinforcement.

In making comparisons consideration should be given to the difference in age of the specimens when tested.

The appearance of the specimens after failure is shown in Figs. 74 to 85, inclusive.

WASHINGTON, April 28, 1920.

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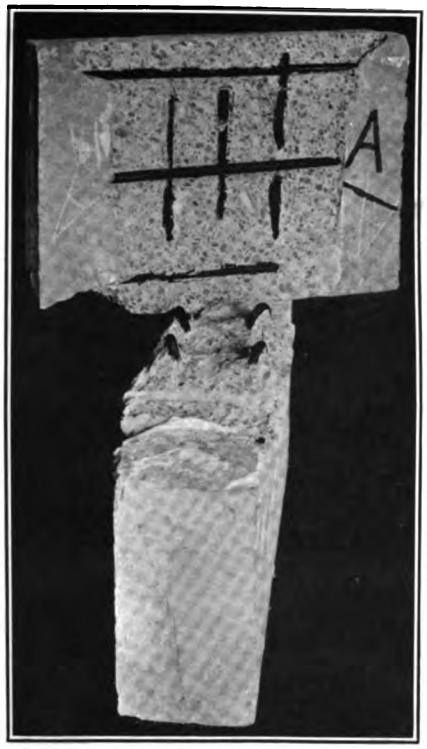

FIG. 74.—*View of specimen A after failure*

FIG. 75.—*View of specimen D after failure*

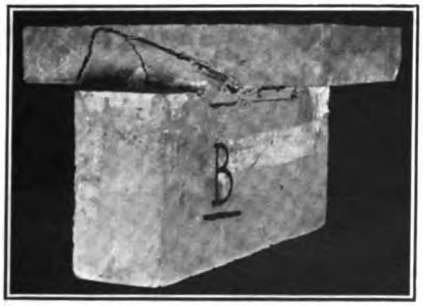

FIG. 76.—*View of specimen B after failure*

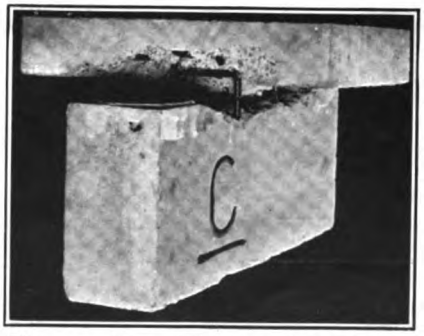

FIG. 77.—*View of specimen C after failure*

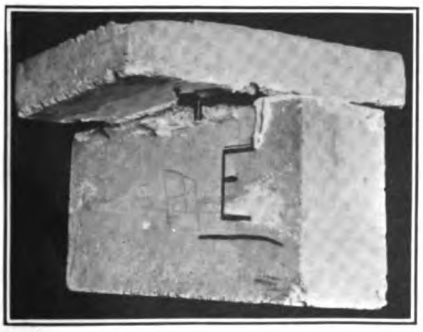

FIG. 78.—*View of specimen E after failure*

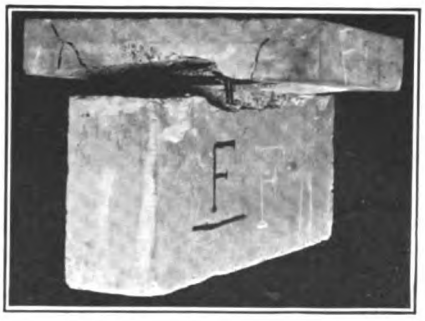

FIG. 79.—*View of specimen F after failure*

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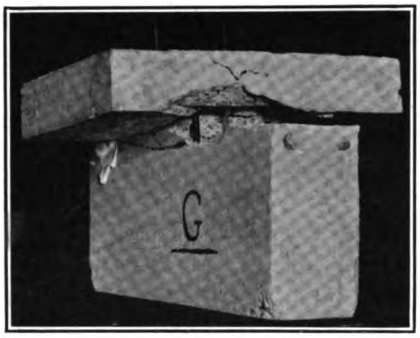

FIG. 80.—*View of specimen G after failure*

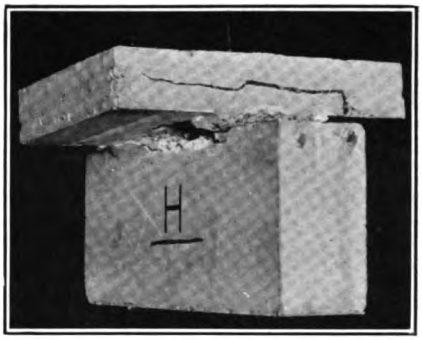

FIG. 81.—*View of specimen H after failure*

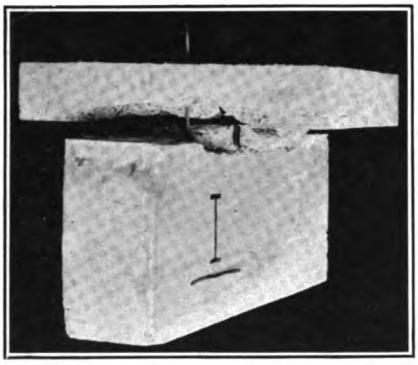

FIG. 82.—*View of specimen I after failure*

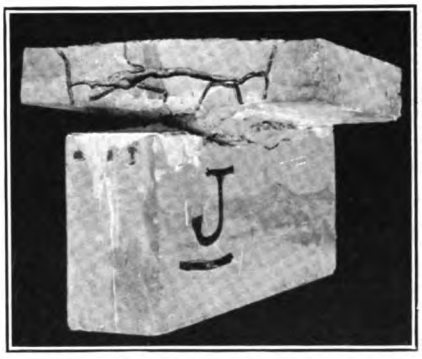

FIG. 83.—*View of specimen J after failure*

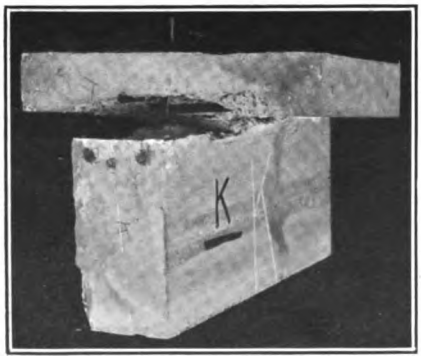

FIG. 84.—*View of specimen K after failure*

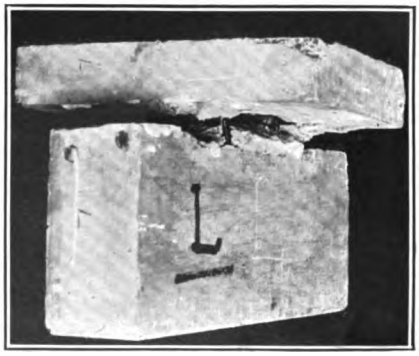

FIG. 85.—*View of specimen L after failure*

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No. 174

EFFECTS OF CAL AS AN ACCELERATOR OF THE HARDENING OF PORTLAND CEMENT MIXTURES

BY

ROY N. YOUNG, Associate Chemical Engineer

Bureau of Standards

OCTOBER 11, 1920

PRICE, 5 CENTS

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EFFECTS OF CAL AS AN ACCELERATOR OF THE HARDENING OF PORTLAND CEMENT MIXTURES[1]

By Roy N. Young

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[1] As explained in the introduction, the effect of calcium chloride in Portland cement mixtures has been quite thoroughly investigated, and the use of hydrated lime in such mixtures is not uncommon in practice. There are no restrictions on the use of these materials in concrete, but their application in the form of calcium oxychloride, or "Cal," has been covered by patents. In view of the importance to users of cement and concrete of knowing the properties of this material and its value as an accelerator, particularly in comparison with calcium chloride, a cooperative investigation was undertaken by the Bureau of Standards and the patentee, the results of which are given in this paper.

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I. INTRODUCTION

A demand for a practical and efficient material for accelerating the hardening of Portland cement mixtures has received considerable attention in recent years. The need for such a material becomes of more consequence with the introduction of more rapid methods in construction and with the increasing amount of concrete repair and replacement work. Serious problems arise when it becomes necessary to interrupt or divert traffic for the duration of time required for concrete to harden sufficiently. The increased cost of lumber for forms has necessitated a more judicious use of such material, and its removal for further use as soon as the strength of the concrete will allow. This applies even more to the use of steel forms, which are rapidly replacing wooden forms in a great many types of concrete construction.

Considerable work toward the development of an accelerator for concrete has been done by the U. S. Bureau of Standards,[2] which found that calcium chloride was the most effective of the various substances tested. Comprehensive tests were made on concrete gaged with solutions up to 10 per cent commercial calcium chloride for the purpose of substantiating the results of the preliminary tests and also to determine if any injurious effects would result at later periods from the use of this accelerator. The increase in strength obtained at the age of two or three days over that of similar concrete gaged with water was from 30 to 100 per cent.[3] At the age of one year there was no indication that the addition of this salt had any deleterious effect on the durability of concrete. The three-year test pieces are now being broken with results showing further evidence that calcium chloride does not effect the ultimate integrity of concrete.

Inasmuch as the use of commercial calcium chloride in concrete is attended by difficulties caused by its highly hygroscopic property and by the handling of the solution, Charles Catlett[4] conceived the idea of introducing the salt by means of Cal, a material by which the above difficulties might be eliminated. Furthermore, Cal possesses possibilities which calcium chloride does not have, owing to the fact that the products of its decomposition by water are calcium chloride and calcium hydroxide, equivalent in effect to a simultaneous addition of calcium chloride and finely divided hydrated lime. Since both calcium

[2] Engineering Record, 74, pp. 266–267; 1916.
[3] Annual Report of the Director of the Bureau of Standards; 1918.
[4] Patent No. 1282188; patent No. 1308932.

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chloride and hydrated lime are being used separately in cement mixtures to produce certain desirable effects, it was not considered that the use of Cal introduces anything new in the making of concrete, excepting the manner in which these materials are added.

Influenced by the above considerations, an investigation was inaugurated by Mr. Catlett, with the cooperation of the Bureau of Standards, for the purpose of studying the effects of Cal on Portland cement mixtures. The most important object was to learn the value of this material as an accelerator of the hardening of concrete.

While a greater part of the work preliminary to the main series of concrete tests deals with the effects of Cal on the early time strengths of cement mixtures under various conditions, it also includes miscellaneous tests bearing on questions which might preclude the use of this material with Portland cement, or limit its use to special mixtures or kinds of construction work.

Acknowledgment is made to G. M. Williams for assistance in planning the concrete tests, to Watson Davis for assistance in conducting these tests, and to A. B. Peck for the microscopic work.

II. NATURE OF CAL

Cal is essentially an oxychloride of calcium. A number of investigators have prepared crystals of this salt by boiling calcium hydroxide in a fairly strong solution of calcium chloride, filtering rapidly, and allowing the filtrate to cool. An analysis by Beesley [5] indicated that the formula was $3CaO.CaCl_2$, $14H_2O$. According to Rose [6] and André,[7] the crystals have the formula $3CaO.CaCl_2$, $16H_2O$. Grimshaw [8] found the constituents in the ratio $3CaO.CaCl_2$, $15H_2O$, but considered the formula $ClCa.O.Ca(OH)$, $7H_2O$ to be simpler and more intelligible.

Calcium oxychloride is readily soluble in water, in which it decomposes into calcium hydroxide and calcium chloride. It is also soluble in alcohol. The salt is acted upon by carbon dioxide, forming calcium carbonate and calcium chloride. Over sulphuric acid, in a vacuum or in the air, it parts with a portion of the water of crystallization. The crystals are needle-shaped and were

[5] Pharmaceutical Journal, 9, p. 568.
[6] Schweigger's Journal für Chemie und Physik, 5, pp. 29–155.
[7] Comptes Rendus, 92, p. 1452.
[8] Chemical News, 80, p. 280.

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prepared by the writer from very minute size up to 1¼ inches in length, depending largely upon the rate at which the solution was cooled. The composition of the crystals made under various conditions was in all cases the same and agreed closely with the formula given by Grimshaw.

Cal is made by pulverizing the product resulting from a mixture of lime, calcium chloride, and water. The material used in the following tests was made by the Mellon Institute at Pittsburgh, Pa., in September, 1918, by mixing together 100 parts hydrated lime, 55 parts commercial calcium chloride, and 50 parts water. During several weeks exposure to the atmosphere it became hard and dry.

There was a question as to whether this material was chiefly a mechanical mixture or a product of chemical reaction. A microscopic examination was made, and it was found to consist of fairly well developed crystals of calcium oxychloride lying in a groundmass of minute irregular crystals which resembled the larger ones in every other respect. There was only a small percentage of calcium hydroxide present. Other samples were prepared by treating separately quicklime and hydrated lime with various concentrations of calcium chloride solutions. These proved to be of the same nature as the Cal just described, with the exception that the larger crystals varied in size and relative number among the different samples.

The first lot of Cal was ground without further drying than that which it received by exposure to the atmosphere. The finished product was a dry, white, rather fluffy powder which analyzed as follows:[9]

	Per cent
Loss on ignition	43.93
Hydrochloric acid insoluble	.15
SiO_2	.71
R_2O_3	.51
CaO	41.15
MgO	3.58
Cl_2	11.47
SO_3	.71
	102.21
O=Cl	2.59
Total	99.62
CO_2	3.78
Water (loss−CO_2)	40.15

[9] The Cal used in the concrete tests was dried and will be described later.

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If Cal is exposed to the air, it gradually takes up carbon dioxide and becomes damp. However this action is comparatively slow and should not cause trouble if the material is handled in the same way as hydrated lime.

A sample of Cal which had been exposed to the laboratory air for eight months and another which had been stored in a cloth bag for about three years were used for treating mortars in compression tests. The results indicated that exposure does not lessen the effectiveness of Cal as an accelerator of the hardening of Portland cement mixtures.

III. EFFECT OF CAL ON THE SETTING TIME OF PORTLAND CEMENT

The effect of Cal on the setting time of normal Portland cements may be seen by examining the results given below. The set was hastened in all cases and to a greater extent by the larger percentage of Cal.

TABLE 1.—The Setting Time of Normal Portland Cement Treated with Undried Cal

[The percentage of water is calculated on the basis of the combined weights of the cement and Cal]

Cement	Ratio (percentage) Cal to cement	Ratio (percentage) water to total mixture	Initial set	Final set
Sample Nos.:			H. M.	H. M.
1	None	23.4	4 0	8 10
	5	21.9	2 15	5 45
	8	21.0	1 50	5 30
2	None	23.2	3 35	6 45
	5	21.7	1 50	4 30
	8	20.8	1 25	3 30
3	None	23.4	3 40	7 0
	5	21.9	2 45	5 45
	8	21.0	2 5	4 25
4	None	24.0	4 40	7 30
	5	22.5	3 0	5 40
	8	21.6	1 50	3 40
5	None	22.5	3 35	6 25
	5	21.0	1 55	2 55
	8	20.1	1 5	1 55

It will be noticed that the percentage of water necessary for normal consistency decreases as the amount of Cal increases. This may be explained in part by the fact that the Cal contains considerable water which is set free upon its decomposition and which is in excess of that necessary to give plasticity to the resulting products.

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Cal was added to a number of quick-setting cements with the result that they were made normal. This may seem contradictory to the results obtained with normal cements as shown above, but it should be remembered that the setting time in such cases is not controlled by Cal alone. Ground Portland cement clinker is usually quick setting, and therefore gypsum is added as a retarder in order to meet commercial requirements. If Cal is added to a finished cement, the effect on the set will be the resultant effect of both Cal and gypsum. The effect of Cal alone on ground cement clinker is as follows:

TABLE 2.—The Setting Time of Ground Cement Clinker Treated with Cal

Cal	Ratio (percentage) water to mixture	Initial set	Final set
		H. M.	H. M.
None	25.0	Flash ..	.....
2 per cent	24.5	1 10	5 0
4 per cent	23.1	3 10	6 10
6 per cent	21.7	2 55	5 45
8 per cent	21.3	2 30	4 20
10 per cent	20.0	1 10	3 25

It will be seen that the maximum retardation is produced by 4 per cent of Cal, and that the retardation diminishes with increasing or decreasing amounts. Hence it seems reasonable that an addition of Cal to a normal cement already containing gypsum should hasten the set in the same way that an addition of Cal to the clinker already containing 4 per cent of Cal will hasten the set. Also, if a cement containing gypsum is quick setting, an addition of Cal in increasing amounts should increase the retardation until a maximum retardation is reached due to the combined effect of Cal and gypsum.

IV. EFFECT OF CAL ON THE SOUNDNESS OF PORTLAND CEMENT

No indication of unsoundness has appeared in normal cement treated with Cal. Unsoundness, as determined by the steam test, may be corrected by the addition of Cal, according to the results of tests given below. This action is probably due to an increased hydration of the free lime before the cement sets.

A very unsound cement was used in a 1:3 mortar which was molded into 2-inch cubes and stored in air after 24 hours. At the end of 8 months, the plain mortar test pieces were entirely

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disintegrated. The test pieces made from the same mortar to which 5 per cent of undried Cal was added were still very firm, although a few disintegration cracks had developed.

TABLE 3.—Effect of Cal on the Soundness of Portland Cement

Cement laboratory No.	Soundness after addition of various percentages of undried Cal			
	0 per cent	2 per cent	5 per cent	8 per cent
42895	Unsound; slightly warped, not hard.	Sound; very slightly warped,	Sound; very slightly warped.	Sound.
42896	do	Unsound; warped, hard.	Sound; very slightly warped. hard.	Sound; very slightly warped, hard.
43015	Unsound; very soft	Unsound; warped, not hard.	Sound	Sound.
43200	do	Sound	do	Do.
5	do		Sound; slightly warped, hard.	Do.

V. EFFECT OF CAL ON THE EARLY COMPRESSIVE STRENGTH OF PORTLAND CEMENT MORTARS

1. WATER STORAGE

A series of tests was made on five different brands of cement, using 1 part cement to 3 parts Standard Ottawa sand. Per centages of undried Cal varying from 0 per cent to 8 per cent of the cement were added to the dry mixtures. Also, mortars were made from each brand of cement by gaging the mixtures with a 4 per cent solution of commercial calcium chloride. This concentration was used for the reason that it was found to be the most satisfactory in tests previously conducted by this Bureau.

Mixes containing additions of Cal up to 5 per cent were, in some cases, gaged with the same quantity of water as was used in the plain mortar for normal consistency. Consequently the excess of water contained in the Cal produced a wetter consistency than normal, and probably resulted in somewhat smaller increases in strength over the plain mortars than would have been obtained if the consistencies had been the same throughout. All other mixes were of normal consistency as calculated from the normal consistencies of the neat mixtures, or as nearly as could be judged by the operators. The results of these tests are shown in Fig. 1. In general, with each increase in the amount of Cal added to the mortar there is an increase in strength at each of the three periods tested.

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2. AIR STORAGE

Some of the above tests were repeated, with the exception that the test pieces were stored in the laboratory air rather than in water. The results of these tests are shown in Fig. 2. It is evident from the much lower strengths of the 28-day test pieces stored in air that there was a very rapid drying out of these test pieces, as might be expected on account of their size. The effect of Cal under the conditions of these tests is remarkable.

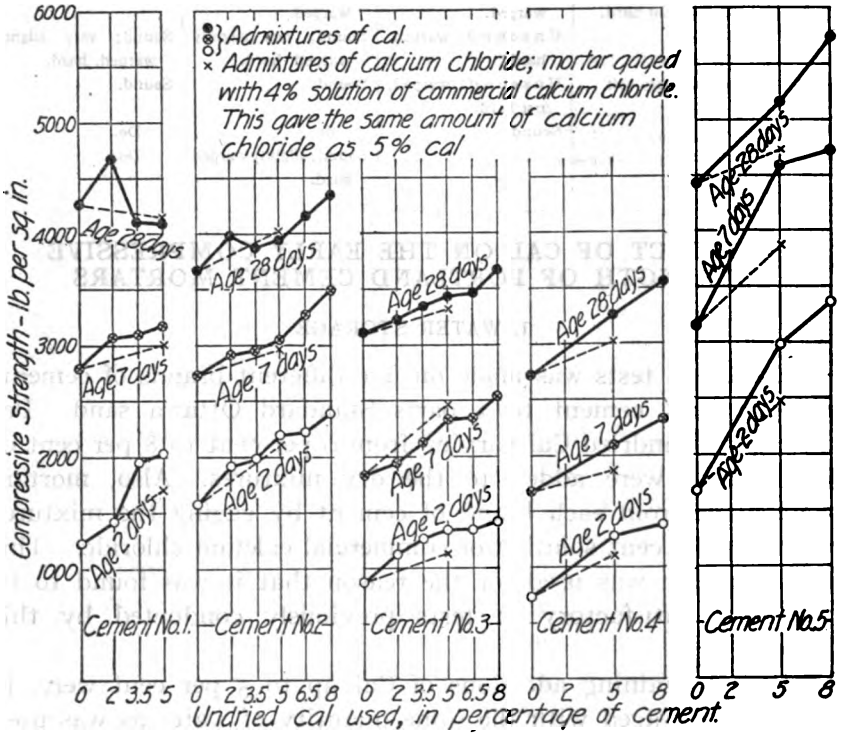
Admixtures of cal.
Admixtures of calcium chloride; mortar gaged with 4% solution of commercial calcium chloride. This gave the same amount of calcium chloride as 5% cal
Compressive Strength - lb. per sq. in.
6000
5000
4000
3000
2000
1000
Age 28 days
Age 7 days
Age 2 days
Cement No. 1.
Cement No. 2
Cement No. 3
Cement No. 4
Cement No. 5
0 2 3.5 5
0 2 3.5 5 6.5 8
0 2 3.5 5 6.5 8
0 2 5 8
0 2 5 8
Undried cal used, in percentage of cement.

FIG. 1.—*Compressive strength of mortar at various ages treated with Cal or calcium chloride and stored in water*

The strength of the treated mortar at 2 days is considerably above that of the untreated mortar at 28 days.

3. COLD STORAGE

It is a well-known fact that the rate of hardening of cement is affected by the temperature conditions. One difficulty arising from this is slow setting and hardening, which often occurs during cold weather. Tests in which the test pieces were stored in a refrigerator at a temperature slightly above freezing indicate

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that such difficulties may be overcome by the use of Cal. Also, Cal may be a means of protection against freezing in that it greatly reduces the critical time during which freezing is injurious.

4. MIXTURES OF CEMENT AND CAL STORED IN AIR

As it might be desirable to mix an accelerator with cement at the mill during the process of grinding or before it is packed, a series of tests was made to determine the effects of air storage on mixtures of cement and Cal. The mixtures in these tests were portions of the ones from which the mortar tests shown in Fig. 1 were made. About 2 kg of each mixture were placed in

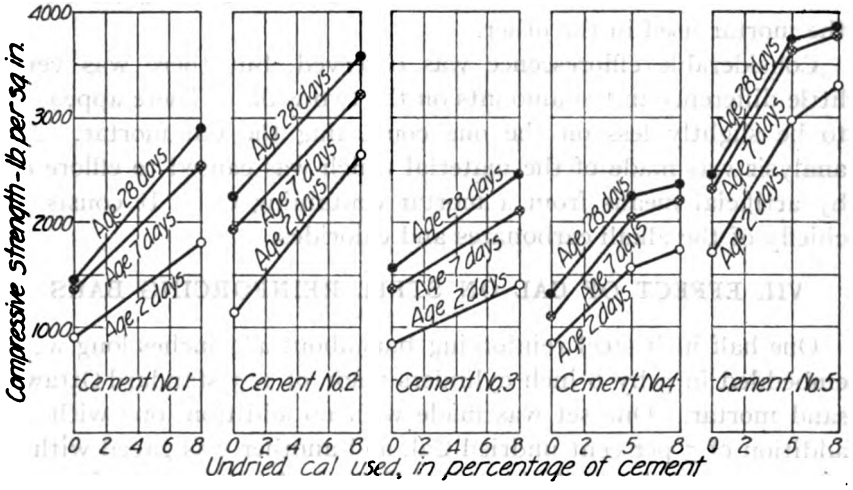

FIG. 2.—*Compressive strength of mortar at various ages treated with Cal and stored in air*

a small canvas bag and stored in the laboratory air. At the end of 90 days mortar cubes were made under as nearly as possible the same conditions as those existing during the former tests.

All samples of cement and mixtures of cement and Cal deteriorated during the storage period. There was a tendency for the mixtures to form lumps and pack. The mortar tests showed no marked increase in strength of mortar containing Cal over that of the untreated. After these tests were started, it was found that the Cal used gives up considerable moisture when exposed to the air, so no doubt that in this instance the moisture given up by the Cal which was mixed with the cement did much toward affecting this deterioration. It was not determined whether the result might be entirely attributed to this loosely

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held water or whether the Cal increased the amount of moisture taken up by the cement from the air. It may be possible that a change in the method of manufacture of Cal will produce a material which can be mixed with cement without ill effects and at the same time retain its efficiency when the mixture is used in concrete.

VI. EFFLORESCENCE

Two small brick walls were built on the roof of a building where they were exposed to the weather. Plain mortar (1 part cement to 3 parts Potomac River sand, by weight) was used in constructing one of the walls, while 8 per cent undried Cal was added to the mortar used in the other.

Considerable efflorescence was observed, but there was very little difference in the amounts on the two walls. There appeared to be slightly less on the one containing the Cal mortar. An analysis was made of the material which was caused to effloresce by artificial means from a mortar containing Cal. It consisted chiefly of the alkali carbonates and chlorides.

VII. EFFECT OF CAL ON STEEL REINFORCING BARS

One-half-inch steel reinforcing bars about 4½ inches long were embedded in 3 by 6 inch cylinders made of 1:3 standard Ottawa sand mortar. One set was made with no addition, one with an addition of 5 per cent undried Cal, and another was gaged with a solution of calcium chloride which furnished an amount of anhydrous calcium chloride equal to that contained in the 5 per cent addition of Cal. These cylinders were stored exposed to the weather. At the end of eight months one test piece from each set was broken, and the only signs of corrosion found were a few rust spots on the bar embedded in mortar containing calcium chloride. Of course, these tests furnish no conclusive information, but it is interesting to note that corrosion had not appeared on the bar in the mortar containing Cal. Such tests should be continued for a number of years and under various moisture conditions.

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VIII. EFFECTS OF CAL AND CALCIUM CHLORIDE ON CONCRETE

1. DESCRIPTIONS AND TESTS OF MATERIALS

(a) CAL

It was found advantageous to drive off a portion of the water in the Cal, and therefore in preparing the lot used in the following tests the lump material was dried. This was done in such a way that it was not exposed to the atmosphere or gases from the burner during the operation. The crushing was done in a small jaw crusher and the grinding in a pebble mill. A sample of about 100 pounds was mixed thoroughly by placing it in the pebble mill lightly loaded with pebbles. The entire quantity was quartered down for the chemical sample. An anlysis of this sample was made several days before the tests were started, and the chlorine content was again determined after all the concrete was mixed, to learn if any change in the percentage, caused by exposure to the air, had taken place during this time. It may be seen by the results given below that equal weights of the material taken at any time during the tests furnished practically equal amounts of calcium chloride.

CHEMICAL ANALYSIS OF CAL

[Laboratory No. 56971]

	Per cent
Loss on ignition	21.42
Insoluble material	.80
Soluble material:	
SiO_2	1.00
R_2O_3	.60
CaO	58.10
MgO	4.75
Cl_2	15.68
SO_3	.91
	103.26
O=C=Cl	3.54
Total	99.72
CO_2	5.12
Water (loss—CO_2)	16.30
Chlorine content after about eight weeks	15.50

(b) CALCIUM CHLORIDE

A solution of calcium chloride was made from commercial calcium chloride and filtered through a finely woven cloth, separating considerable insoluble matter. After a sample was taken for chemical analysis, the solution was kept in an air-tight container

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until used. The samples for chemical analysis of this calcium chloride solution were taken at the same time as those for the analysis of Cal. The analysis of this solution follows:

RESULTS OF ANALYSIS OF CALCIUM CHLORIDE SOLUTION

[Specific gravity at 24° C=1.338]

	Per cent
Ca	12.16
Cl_2	22.31
Mg, less than	.10
Chlorine content after the concrete mixing was completed	22.42

(c) CEMENT

On account of the somewhat varying effectiveness of Cal on different brands of cement as indicated by the mortar tests, it was decided that no less than three brands should be used in this series. These brands were chosen from those used in the mortar tests for the reason that they had been found to fairly well represent the range of Portland cements with regard to early time strengths and that they could be purchased on the market at Washington, D. C.

The entire lot of each brand was thoroughly mixed on canvas and then placed in metal containers until used. The chemical and physical tests of these cements are given in Table 4.

TABLE 4.—Chemical and Physical Tests of Cements

Nature of determination	Test results for laboratory numbers—		
	45003	45076	45101
Chemical:	Per cent	Per cent	Per cent
Loss	2.01	2.43	1.76
Insoluble residue	.20	.17	.40
Silica	19.78	22.39	20.72
Oxide of iron	3.40	2.20	1.90
Oxide of alumina	7.26	5.58	5.82
Lime	62.56	60.00	63.48
Magnesia	1.72	4.07	4.02
Sulphuric anhydride	1.91	1.78	1.63
Sodium oxide	.....	.87	.43
Potassium oxide	.....	.42	.38
Physical:			
Water used—			
Neat	23.0	23.0	23.0
Mortar	10.3	10.3	10.3
Fineness (200-mesh sieve)	84.5	81.6	87.1
Time of set—	Hr. Min.	Hr. Min.	Hr. Min.
Initial	2 55	2 55	3 45
Final	6 30	6 00	6 55
Soundness	O. K.	O. K.	O. K.
Tensile strength—	Lbs./in.²	Lbs./in.²	Lbs./in.²
7 days	282	300	318
28 days	370	313	455

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(*d*) AGGREGATE

Potomac River aggregate was used in this series and was purchased in 5-ton lots at Washington, D. C. The gravel was washed, dried, and stored in sacks until used. To insure uniformity, just before each day's run of mixing, a sufficient amount of gravel for all mixes for one proportion and for one brand of cement was mixed by shovel on a concrete floor and resacked in lots of accurate weight for one batch. The sand was treated in the same manner excepting that it was not washed, as it was in good condition when received. The results of the physical tests of these materials are given below. The weights per cubic foot used in proportioning were 95 per cent of the weight per cubic foot of packed material. A metal cubic-foot measure was used in this determination.

WEIGHTS PER CUBIC FOOT

	Pounds
Sand	120
Gravel	114

TABLE 5.—Mechanical Sieve Analysis of Aggregate

Sieve No.	Size opening	Per cent passing		Sieve No.	Size opening	Per cent passing	
		Sand	Gravel			Sand	Gravel
	Inches				Inch		
	1½	.....	100	4	0.185	95	1
	1	.....	91	8	.093	81	.....
	¾	.....	71	14	.046	69	.....
	½	.....	51	28	.0232	51	.....
	¼	.....	26	48	.0116	15	.....
8	.263	100	8	100	.0058	4	.....

2. METHODS AND APPARATUS

(*a*) PROPORTIONING

The proportioning of the cement, sand, and gravel was by volume, but for the sake of convenience and accuracy the materials were measured by weight. The cement was assumed to weigh 94 pounds per cubic foot.

In all mixes in which a direct comparison was made between Cal and calcium chloride, an exact weight of the calcium chloride solution was used which contained an equivalent weight of chlorine as added by means of the Cal.

(*b*) CONSISTENCY

The consistency of the various mixes was measured by means of the "flow table," which has been developed by the Bureau of Standards.[10] It is believed that this method is the most satis-

[10] Williams, Engineering News Record, 84, p. 1044.

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factory of any so far devised for determining the relative "flowability" of concrete. The method is sensitive through a wider range of consistencies than the slump method, and the results are consistent and reliable.

Two consistencies were chosen as standards in the concrete tests. The 1 : 1½ : 3 concrete was rather stiff and represented the type of concrete often used in road construction. All batches of this proportion were made to have a relative flowability as near 150 as possible.

The 1 : 2 : 4 concrete was considerably wetter and was of such consistency as is often used in reinforced-concrete construction. The relative flowability sought in this case was 200.

From two to three flow tests were made on each batch in order that very nearly the desired consistency might be produced before the final test was made. This final test was to be made 10 minutes after the water was added to the dry mixtures.

(c) MIXING

In the preliminary tests on concrete contradictory results were obtained under certain conditions in that the strengths of the treated concrete at seven days were lower than the corresponding strengths of the untreated concrete. Considerable work was done to determine the cause of this before it became evident that the different method used in mixing the concrete was the source of the difficulty. In every instance the mortars were mixed by kneading with the hands, and the batches were sufficiently small to allow thorough mixing within one and one-half minutes. The batches of concrete were considerably larger and were mixed by turning with a trowel. Consequently they did not receive as vigorous or thorough mixing as applied to the mortars. When these concrete mixes were duplicated and mixed by machine the results were of the same order as those obtained in the mortar tests. No further investigation was made in this direction, but it is believed that the results obtained with the hand-mixed concrete were exceptional.

In the following tests the mixing was done in a model 15 Wonder mixer. Sufficient material was used in each batch to provide some in excess of the amount necessary for six 6 by 12 inch cylinder test pieces. After the dry materials were placed in the mixer a part of the water was added and the mixer was started. More water was then added until about the proper consistency was obtained. At the end of 2 minutes from the time the first water

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was added the mixer was stopped and a "flow" test was made. When only two flow tests were made within 10 minutes after the water was added the mixer was run 45 seconds just previous to the making of the final flow test. Later three flow tests were made instead of two, and the mixer was run for 30 seconds before each of the last two tests. This additional mixing of all batches was necessary in order that water might be added if needed to increase the flowability to the desired degree and at the same time that all batches might be given an equal amount of mixing.

(*d*) MOLDING AND CAPPING

All test pieces in this series were 6 by 12 inch cylinders. The molds used were made from wrought-iron pipe cut in lengths of 12 inches and slit perpendicularly to the axis of the cylinder. This slit was kept closed during molding by bolts through angles riveted to the cylinder on each side of the slit. The molds were filled in three layers, each layer being puddled with a five-eighth-inch steel rod. The bottom caps were formed by placing the molds on a layer of stiff neat cement which was spread about one-half inch thick on sheet iron a short time before molding the test pieces. The molds were filled to less than one-fourth inch from the top. After about an hour the filling was completed with a thin neat cement paste and smoothed off with a trowel. This method gave good solid caps which required very little grinding to produce a plane bearing surface.

(*e*) STORAGE

At the end of 24 hours the test pieces were removed from the molds. Three of the test pieces from each batch were stored in damp sand, and the remaining three were stored in the air of the laboratory. Hence, the results obtained under the two storage conditions are strictly comparable with respect to materials, quantity of mixing water, etc. The daily average room temperatures where test pieces were stored in air gradually lowered throughout the storage period from about 72° F. to about 60° F. The temperature of the damp sand averaged about 5° F. below these temperatures at the beginning of the tests, but toward the end the temperatures of the two storages were about the same. The average relative humidity was approximately 65 per cent.

(*f*) TESTING

Specimens 62–1–1 to 62–3–3 and 62–13–1 to 62–15–3,[11] inclusive, were tested on a 200 000-pound Riehle testing machine. All others

[11] The numbering system is merely a local means of identifying the test pieces.

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were tested on a 100 000-pound Riehle machine. A single thickness of blotting paper was placed between each end of the specimen and the bearing plate. The test piece was centered on a spherical bearing block which was then shifted into place. The upper surface of the test piece was fitted squarely against the upper bearing surface by turning the test piece and upper part of the block horizontally first in one direction and then in the other while the movable head was being lowered onto the test piece.

3. RESULTS OF STRENGTH TESTS

The results of the strength tests are given in Table 7.

The most important facts deduced from these data are as follows:

1. In all cases, the strength of the treated concrete was greater than that of the corresponding untreated concrete at the two and seven day periods.

2. The strength of the treated concrete was practically the same as that of the untreated at the 28-day period.

3. The effect of Cal at the 2-day period was very nearly the same for all cements, proportions, and conditions of the tests. This may be seen by examining the data given below.

TABLE 6.—Percentage Increase in Strength of Concrete Due to Cal at the Two-Day Period

Resulting from—	Maximum	Minimum	Average
Cement No. 45003	75	60	66
Cement No. 45076	80	55	65
Cement No. 45101	66	52	58
Proportion 1 : 1½ : 3, stiff consistency	61	52	58
Proportion 1 : 2 : 4, wet consistency	80	52	68
Stored in damp sand	80	52	65
Stored in air	69	52	61

4. The results produced by Cal and calcium chloride are practically the same.

Since the primary reason for using an accelerator in Portland cement mixtures is to produce a given strength in less time than ordinarily required and thereby effect a saving, it is important to express laboratory tests on this basis. If time-strength curves of both the untreated and corresponding treated mixtures are plotted together, it is very convenient to compare the ages at which they have equal strengths. In this series of concrete tests, the strength of the treated concrete at two days was equal to that of the untreated at from three to four and one-half days. Mortar tests indicated that more marked results may be obtained by increasing the percentage of Cal.

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Although very uniform results were obtained under various conditions at the 2-day period, there was considerable variation at the 28-day period. Furthermore, Cal generally increased the strength of mortars at the 28-day period, while it seemed to have little or no effect on the concrete at this age. A number of mortar tests were made with variations in proportions and percentages of Cal to cement and Cal to water, with the thought that they might determine the cause of these irregularities, but no conclusive results were obtained.

TABLE 7.—Compressive Strengths of Concrete to Which Cal and Calcium Chloride Have Been Added

[Additions of Cal throughout this series were 5 per cent of the cement. Cal and calcium chloride additions contained equal amounts of chlorine. Test pieces were 6 by 12 inch cylinders]

CEMENT NO. 45003

Batch	Addition	Proportion	Relative flowability	Water, per cent	Compressive strength in pounds per square inch					
					Stored in damp sand			Stored in air		
					2 days	7 days	28 days	2 days	7 days	28 days
62-1-1	None	1 : 1½ : 3	150	7.88	870	2390	3491	806	2370	3662
2	do	1 : 1½ : 3	156	7.88	915	2390	4035	882	2450	3351
3	do	1 : 1½ : 3	155	7.88	915	2290	3735	916	2190	3508
Average			155	7.88	900	2337	3754	868	2337	3507
62-2-1	Cal	1 : 1½ : 3	149	7.88	1488	3030	3775	1290	2470	3379
2	do	1 : 1½ : 3	148	7.88	1432	2770	3788	1598	2610	3600
3	do	1 : 1½ : 3	149	7.88	1435	2650	3687	1279	3000	3689
Average			149	7.88	1450	2817	3617	1390	2693	3556
62-3-1	$CaCl_2$	1 : 1½ : 3	148	7.88	1686	2700	3471	1515	2610	3339
2	do	1 : 1½ : 3	149	7.88	1347	2790	4089	1540	2790	3665
3	do	1 : 1½ : 3	154	7.88	1600	2790	3390	1300	2790	3684
Average			152	7.88	1540	2760	3650	1450	2730	3536
62-4-1	None	1 : 2 : 4	189	8.52	338	1222	2212	322	1090	1782
2	do	1 : 2 : 4	212	8.52	341	1148	2314	357	1109	1967
3	do	1 : 2 : 4	201	8.52	384	1236	2303	392	1374	1976
Average			201	8.52	354	1202	2278	357	1191	1908
62-5-1	Cal	1 : 2 : 4	190	9.28	593	1329	1765	601	1242	1612
2	do	1 : 2 : 4	200	8.89	674	1340	1750	553	1295	1591
3	do	1 : 2 : 4	213	8.89	586	1455	1976	656	1330	1612
Average			201	9.02	618	1375	1830	603	1289	1605
62-6-1	$CaCl_2$	1 : 2 : 4	217	8.58	686	1480	2112	636	1439	2012
2	do	1 : 2 : 4	196	8.28	740	1605	1943	756	1562	1998
3	do	1 : 2 : 4	200	8.28	716	1515	2282	636	1491	1800
Average			206	8.38	714	1533	2112	676	1497	1937

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TABLE 7.—Compressive Strengths of Concrete to Which Cal and Calcium Chloride Have Been Added—Continued

CEMENT NO. 45076

Batch	Addition	Proportion	Relative flowability	Water, per cent	Compressive strength in pounds per square inch					
					Stored in damp sand			Stored in air		
					2 days	7 days	28 days	2 days	7 days	28 days
62–7–1	None	1 : 1½ : 3	156	8.14	389	952	2362	396	1045	1733
2	do	1 : 1½ : 3	150	7.94	452	1141	2231	408	1117	1855
3	do	1 : 1½ : 3	151	7.94	428	1041	2327	395	1050	1822
Average			152	8.01	423	1045	2307	400	1071	1803
62–8–1	Cal	1 : 1½ : 3	147	8.21	666	1226	2228	621	1385	1922
2	do	1 : 1½ : 3	152	8.21	704	1183	2217	672	1363	2029
3	do	1 : 1½ : 3	142	8.21	598	1176	2257	643	1315	1883
Average			147	8.21	656	1196	2234	645	1354	1945
62–9–1	$CaCl_2$	1 : 1½ : 3	148	8.02	676	1329	2362	690	1400	2093
2	do	1 : 1½ : 3	155	8.02	661	1290	2292	600	1383	2030
3	do	1 : 1½ : 3	161	8.02	617	1277	2041	684	1403	1971
Average			155	8.02	651	1279	2232	658	1395	2031
62–10–1	None	1 : 2 : 4	188	8.80	132	393	1290	146	406	748
2	do	1 : 2 : 4	198	8.80	138	416	1145	141	393	647
3	do	1 : 2 : 4	205	8.80	113	346	1131	147	445	739
Average			197	8.80	128	385	1189	145	415	708
62–11–1	Cal	1 : 2 : 4	200	9.04	229	526	1146	228	516	873
2	do	1 : 2 : 4	210	9.04	252	539	1020	262	569	842
3	do	1 : 2 : 4	196	9.04	209	524	1223	230	633	928
Average			202	9.04	230	530	1130	240	573	878
62–12–1	$CaCl_2$	1 : 2 : 4	201	8.80	274	572	993	279	534	870
2	do	1 : 2 : 4	196	8.80	218	533	1141	251	634	893
3	do	1 : 2 : 4	201	8.80	238	544	1139	229	525	854
Average			199	8.80	243	550	1091	253	564	872
Cement No. 45101										
62–13–1	None	1 : 1½ : 3	156	7.94	1146	2900	3660	1184	2375	3330
2	do	1 : 1½ : 3	149	7.75	1306	2742	4060	1175	2258	3350
3	do	1 : 1½ : 3	152	7.75	1212	2640	3900	1177	2347	3295
Average			152	7.81	1221	2761	3873	1179	2327	3325
62–14–1	Cal	1 : 1½ : 3	152	8.06	1875	3133	3905	1995	2970	3635
2	do	1 : 1½ : 3	150	8.06	1800	2955	3885	1801	2747	3525
3	do	1 : 1½ : 3	149	8.06	1887	2994	.....	1850	2631	3810
Average			150	8.06	1854	3027	3895	1882	2783	3657

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TABLE 7.—Compressive Strengths of Concrete to Which Cal and Calcium Chloride Have Been Added—Continued

CEMENT NO. 45101—Continued

Batch	Addition	Proportion	Relative flowability	Water, per cent	Compressive strength in pounds per square inch					
					Stored in damp sand			Stored in air		
					2 days	7 days	28 days	2 days	2 days	28 days
62-15-1	$CaCl_2$...	1 : 1½ : 3	149	7.75	2020	3299	3845	1981	3061	4215
2	...do....	1 : 1½ : 3	150	7.75	1835	2982	4260	1973	2882	3720
3	...do....	1 : 1½ : 3	150	7.75	1943	2880	3940	1775	2800	3760
Average			150	7.75	1935	3034	4015	1910	2914	3898
62-19-1	None....	1 : 2 : 4	200	8.67	535	1551	2757	457	1177	1808
2	...do....	1 : 2 : 4	199	8.38	504	1510	2422	524	1225	1965
3	...do....	1 : 2 : 4	196	8.92	537	1475	2523	476	1145	1739
Average			198	8.66	525	1512	2567	486	1182	1837
62-20-1	Cal....	1 : 2 : 4	195	8.68	912	.....	2483	774	1448	2090
2	...do....	1 : 2 : 4	194	8.68	862	1554	2418	738	1504	1835
3	...do....	1 : 2 : 4	198	8.68	837	1648	2240	702	1371	2021
Average			196	8.68	870	1601	2380	738	1441	1982
62-21-1	$CaCl_2$...	1 : 2 : 4	196	8.37	872	1694	2490	868	1434	2162
2	...do....	1 : 2 : 4	204	8.72	795	1782	2600	784	1578	1880
3	...do....	1 : 2 : 4	196	8.56	754	1654	2205	780	1458	1888
Average			199	8.55	807	1710	2432	811	1490	1977

IX. EFFECT OF CAL ON CONSISTENCY AND WORKABILITY OF CEMENT MIXTURES

The way in which Cal affects consistency depends largely upon the amount of water which it contains. When Cal containing 40 per cent water was added to cement or mortar, an appreciably wetter mix was the result. Cal containing 16 per cent water had no effect on the consistency or flowability of mortars, but it appeared to increase the plasticity or ease in working the mixtures with a trowel. In the concrete tests a very small increase in percentage of water was necessary in the mixes containing Cal over that added to the untreated mixtures in order to produce the same relative flowability. This was probably due to the fact that Cal causes earlier stiffening or setting and that the flowability tests were made 10 minutes after the water was added. This earlier stiffening is not objectionable if the concrete is placed shortly after mixing. On the other hand, it is at times very desirable, especially in construction requiring a finished surface, such as floors.

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Cal was used in a concrete floor topping alongside of topping containing no accelerator, for the purpose of observing the effect under actual field conditions. The treated topping was ready for the finishing operation in about one-half the time required by the untreated, as judged by the finisher. This agreed very closely with the results of the laboratory tests on the rate of setting of the two mixes by means of the flow table.

X. SUMMARY

Cal is a material obtained by pulverizing the dried or undried product resulting from a mixture of either quicklime or hydrated lime, calcium chloride, and water. It is a dry white powder which may be handled in much the same way as hydrated lime, and with the same general keeping qualities. It is much more convenient to handle and use in making concrete than calcium chloride, either fused or in concentrated solution. Upon exposure to the air Cal gradually takes up carbon dioxide and becomes somewhat damp. However, tests indicate that long exposure does not affect its action as an accelerator of the hardening of Portland cement mixtures.

The setting of normal Portland cement mixtures may be hastened by Cal to an extent which is very desirable in concrete construction requiring a finished surface. The finishing operation may proceed with much less delay after the concrete has been placed, which should result in cutting down overtime labor. This hastening of the set is not objectionable in any type of construction providing the concrete is placed soon after it is gaged with water. It is believed that Cal increases the workability of Portland cement mixtures. However, no attempt was made to measure the extent of this effect, because up to this time no satisfactory test has been developed for measuring the workability of Portland cement mixtures.

Unsound cements may be greatly benefited or made sound by an addition of Cal. This effect was produced in neat pats subjected to the steam test and in mortar test pieces stored in air.

Limited tests indicate that quicksetting cements, either fresh or having become quicksetting on aging, may be made normal by the addition of Cal as used in cement mixtures.

There was no indication that the amount of efflorescence appearing on the surface of cement mixtures exposed to the weather is increased by the use of Cal.

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The series of tests on the effect of Cal on steel reinforcing bars which were embedded in a thin layer of Cal cement mortar and exposed to the weather for eight months without showing any sign of corrosion, while favorable, is too limited to give assurance that corrosion will not occur under these conditions. Caution should, therefore, be exercised in the use of Cal in concrete containing steel reinforcement when the concrete is to be freely exposed to the weather or excessive dampness. It is believed that no bad effect will be produced in ordinary building construction.

Undried Cal mixed with Portland cement causes greater deterioration in the quality of the cement during storage than that which ordinarily takes place. Therefore, Cal should be added to the concrete materials during the mixing operation, preferably before the water is added.

All Portland cement mixtures treated with Cal attained greater strength at the 2-day and 7-day periods than the corresponding untreated mixtures. The percentage increase in the strength of mortar at the 2-day period obtained by an addition of 5 per cent Cal to cement ranged from 40 to 140. The strength of the treated mortar at two days was equal to the strength of the untreated mortar at three and one-half to eight days. These calculations are made from the results of tests in which the test pieces were stored in water, damp sand, or damp closet.

Treated mortars stored in the laboratory air attained at 2 days strength greater than that of the untreated mortars at 28 days. This was due to the rapid drying out of the small test pieces and the comparatively slow rate of gain in strength after the 2-day period. However, this indicates that Cal is especially advantageous in cement mixtures which are necessarily subjected to any drying-out action.

The increase in strength produced by 5 per cent Cal in concrete mixtures at the 2-day period ranged from 52 to 135 per cent, and the strength of the treated concrete at the 2-day period was equal to that of the untreated at from three to four and one-half days. On an average this represents a saving of approximately one-half the time in operations which are dependent upon the strength of the concrete at early periods. The effect of the air storage in the concrete tests was lessened in degree, owing to the high relative humidity which existed throughout the storage period.

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It should be remembered that the increase in strength resulting from an addition of 5 per cent Cal does not represent the maximum which may be obtained except in very rich mixes. As much as 15 per cent Cal was used in mortar tests, giving an increase of 220 per cent at the 2-day period.

The general effect of Cal on Portland cement mixtures is the same as might be expected from the use of equivalent amounts of hydrated lime and calcium chloride.

The 3-year tests by the Bureau of Standards on concrete gaged with a solution of calcium chloride are sufficient grounds for believing that the addition of Cal will not injuriously affect the ultimate strength and integrity of Portland cement concrete.

WASHINGTON, May 26, 1920.

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No. 175

POURING AND PRESSURE TESTS OF CONCRETE

BY

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OCTOBER 11, 1920

PRICE, 5 CENTS
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POURING AND PRESSURE TESTS OF CONCRETE

By W. A. Slater and A. T. Goldbeck

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INTRODUCTION

This technologic paper presents the results of two minor investigations carried out at the request of the concrete ship section of the Emergency Fleet Corporation. They were undertaken with the object of obtaining data useful in design under the special conditions involved in the construction of concrete ships.

I. POURING, CONSISTENCY, AND COMPRESSIVE-STRENGTH TESTS OF CONCRETE IN CYLINDERS AND SLABS

1. POURING TEST

The object of the pouring test was to determine to what extent a satisfactory job of concreting could be secured under conditions likely to exist in field work during the construction of concrete ships. To be satisfactory there should be no voids in the finished concrete. The work would be expected to have a smooth surface finish, and the concrete should have strength commensurate with the mix and should be of uniform quality. This test was made in the concrete laboratory of the Bureau of Public Roads, Arlington, Va., on May 4, 1918.

The pouring specimen used was a slab 6 feet high, 6 feet wide, and 4 inches thick, having a mass of reinforcement designed to reproduce conditions such as would occur in pouring the shell of a

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reinforced concrete ship. Fig. 1 shows the reinforcement assembled and in position with one side of the form temporarily removed.

Wedge-shaped strips cut from 1¼-inch square stock were attached to both of the side forms horizontally at intervals of 12 inches, marking V-shaped grooves along which the slab was later broken into sections for testing transversely. The wedge strips also served the purpose of choking the opening through which the concrete had to pass and therefore made pouring very much more difficult. The percentage of tension reinforcement was taken large enough to insure failure of the concrete in compression in the transverse tests of the slab sections, unless the strength of the concrete, as computed by the parabolic formula, should exceed 3500 pounds per square inch.

An Austin cube mixer having a rated capacity of 1 cubic yard per batch was used for mixing the concrete for this test. The concrete was proportioned by volume, using 1 part of a standard Portland cement, two-thirds part of Potomac River sand, and 1⅓ parts of gravel. The gravel was graded in size from one-fourth to one-half inch. The amount of water used in mixing was 12.5 per cent of the combined weight of the dry materials. The materials were weighed, and the proportions by volume, 1 : ⅔ : 1⅓, were computed from the weights.

Four small batches of concrete were mixed to pour the specimen. Reference to Table 1 shows that there was some variation in the consistency in the different batches. The accidental loss of an unmeasured but small quantity of water during the mixing of the first batch accounts for a stiffness in that batch somewhat greater than that of the succeeding batches.

The cement and aggregate were mixed dry for about 30 seconds after which the water was added and the mixer was run for 1 minute and 45 seconds before dumping the first wheelbarrow load of concrete. It was generally 1 minute more or longer before the remainder of the batch had been removed from the mixer. At the best, an appreciable amount of concrete remained in the mixer from one batch to the next.

The concrete in the lower half of the slab was placed in the form by means of shovels and trowels through an opening in the side of the form 10 inches high and with its lower edge 3 feet 1 inch above the base of the form. The location of this opening is indicated in Fig. 2 by the words "window panel." The form was vigorously pounded with heavy hand hammers throughout the pouring. Air bubbles were released by the pounding and

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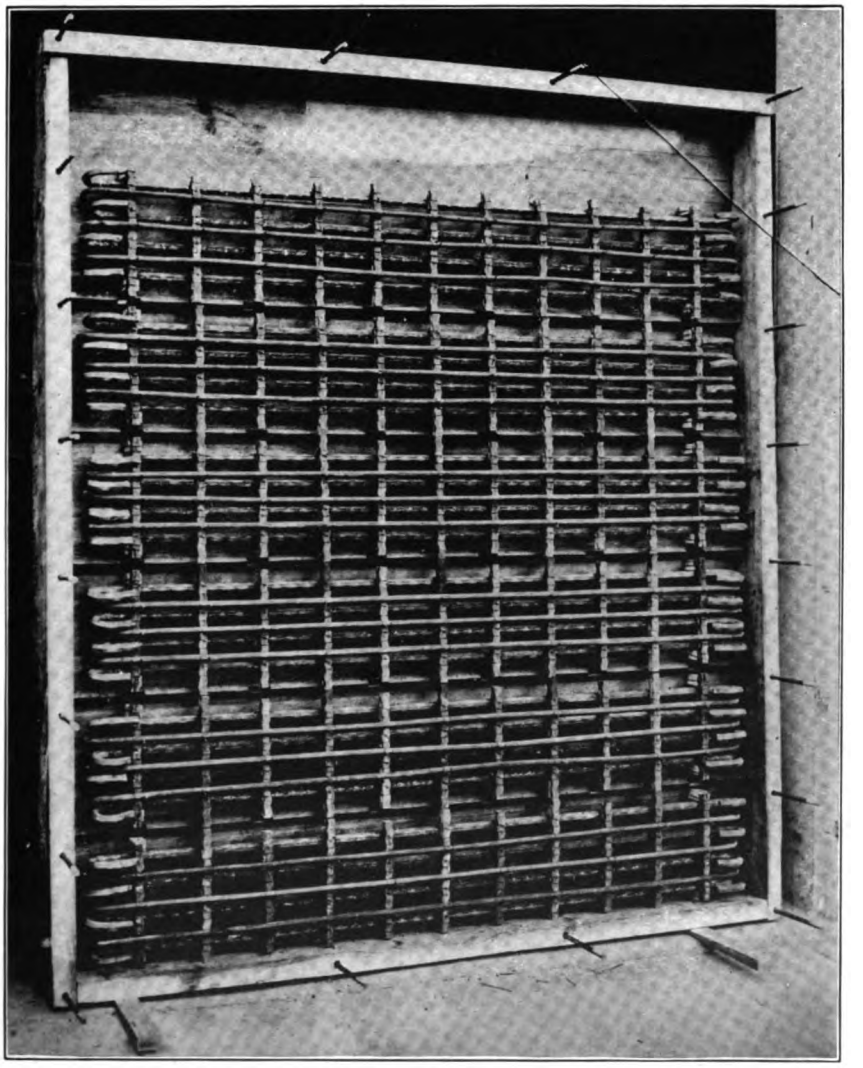

FIG. 1.—*Assembled reinforcement and form for specimen used in pouring test*

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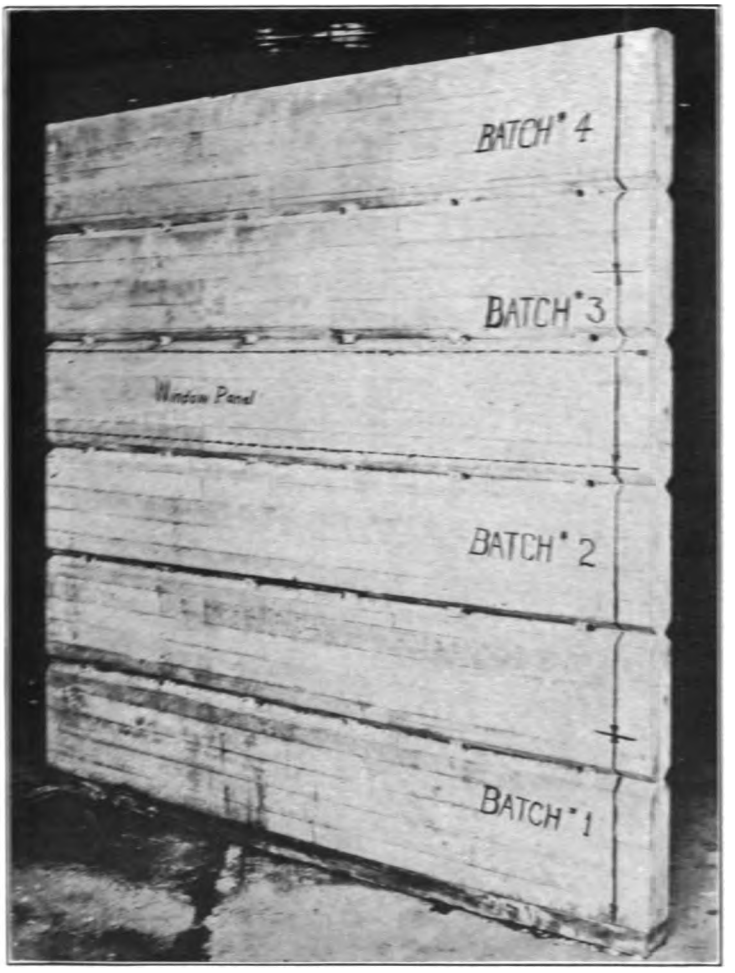

FIG. 2.—*Pouring-test specimen removed from form*

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rose to the surface of the concrete. The rising of air bubbles continued for a considerable length of time, and had not entirely stopped when pouring was completed and the pounding was discontinued.

During the pouring of the slab an observation was made to find the angle of repose of the concrete. The concrete was poured into the form at one point and allowed to flow toward both ends, assisted to some extent by the pounding outside the form. It was found that the slope of the concrete surface was 30° or more above the horizontal.

In order to check the consistency of the mixture used in the pouring tests, four control cylinders were made. The tests for consistency were made by filling 6 by 12 inch cylindrical molds with concrete taken from the wheelbarrow as it was being wheeled to the form for pouring the slab. The molds were placed on a smooth, clean portion of the concrete floor which had previously been dampened. The molds were then lifted at a uniform rate in a line as nearly vertical as possible. This allowed the plastic concrete to run out into a puddle on the floor. The diameter of the puddle thus formed was taken as a measure of the consistency. After the diameter of the puddle had been measured, the concrete of the control specimen was replaced in the mold and kept for a compression test as a control of the strength of the concrete which went into the slab. The cylinders were removed from the molds after 24 hours and stored under damp burlap until tested. Table 1 gives the consistency readings for the control specimens, and also the compressive strength when tested at the age of 31 days.

TABLE 1.—Consistency and Compressive Strength of Control Cylinders

[Water 12.5 per cent of combined weight of dry materials]

Cylinder No.	Batch No.	Barrow No.	Consistency reading[a]	Maximum load	Compressive strength
			Inches	Pounds	Lbs./in.²
1	(b)	.....	16¼	103 940	3670
2	1	3	10	.....	.....
3	2	2	16	.....	.....
4	2	3	13½	102 570	3620
5	3	1	12¾	.....	.....
6	3	2	11	95 000	3350
7	4	1 and 2	13	114 000	4030

[a] The consistency reading is the diameter of the puddle of concrete remaining after lifting the 6 by 12 inch cylindrical mold.

[b] This specimen was made from a hand-mixed batch just large enough for the specimen, previous to starting the pouring test.

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The concrete at the top of the slab was struck off even with the edge of the form in order to provide a means of measuring the amount of settlement during setting. Later observations showed that a settlement of about one-half inch had occurred in a total depth of 6 feet.

One side form was removed the second day after pouring in order to examine the surface of the specimen. This surface is shown in Fig. 2, and its appearance indicates that the pouring was satisfactorily completed. A smooth surface was obtained at all points on the face of the slab. Even under the wedge-shaped strips used to separate the individual slabs only one place was observed where a perfect filling was not obtained. This imperfection was so slight, however, that it is not noticeable in the photograph.

2. STRENGTH TESTS

It was desired to see if any variation existed in the strength of the concrete caused by differences in position in the specimen. Accordingly the specimen was divided into shallow beams for testing separately under transverse loading. A large amount of steel was used for tension reinforcement, with the intention of forcing failure by compression in the concrete. In spite of that provision all of the specimens failed in tension, and the full strength of the concrete was not developed.

Six sections were made by dividing the specimen into beams 12 inches wide and 6 feet long. This separation was made at an age of about 3 days by breaking the slab along the lines of the V-shaped grooves shown in Fig. 2. The beams were numbered from 1 to 6, inclusive, starting with the bottom of the specimen as poured. They were stored under wet burlap and were tested at an age of 31 days.

The reinforcement in each shallow beam consisted of 4 five-eighths inch square corrugated bars in the bottom and 3 one-half inch plain round bars in the top. The top and bottom layers of reinforcement were separated by 11 transverse three-fourths inch square spacing bars 12 inches in length. The amount of bottom reinforcement varied from 4 to 4.7 per cent, on account of the variation in thickness caused by uneven spreading of the form during the pouring. The beams were tested on a span of 5 feet 6 inches, with the load at two points 12 inches apart and each 6 inches from the center of the span. All bearing blocks were bedded in plaster of Paris. Deflections were measured at midspan by means of an Ames dial reading to 0.001 inch.

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The measured thickness of the sections into which the slab was divided was greater than the nominal dimension of 4 inches, indicating that there was a slight bulging of the forms due to the pressure of the concrete.

The compressive stress in the concrete at the maximum load, as computed by the parabolic formula, exceeded the assumed value of 3500 pounds per square inch, and none of the shallow beams failed by compression in the concrete. Table 2 gives the results of the beam tests.

TABLE 2.—Results of Shallow Beam Tests

Beam No.	Measured thickness	Depth to tension steel[a]	Tension reinforcement	Maximum load	Deflection at maximum load	Compressive stress at maximum load f_c		Tensile stress at maximum load, straight-line formula f_s	Ratio computed stress at maximum load to working stress	
						Parabolic formula	Straight-line formula		Compression[b] $\frac{f_c}{1500}$	Tension[c] $\frac{f_s}{16\,000}$
	Inches	Inches	Per ct.	Pounds	Inch	Lbs./in.²	Lbs./in.²	Lbs./in.²		
1	4.13	2.75	4.7	10 300	0.99	4460	6320	40 600	4.22	2.54
2	4.60	2.99	4.4	12 580	.91	4770	6630	45 400	4.42	2.84
3	4.88	3.13	4.2	11 630	.68	4080	5650	40 200	3.76	2.51
4	5.19	3.28	4.0	13 290	.98	4240	5950	43 500	3.96	2.72
5	5.13	3.25	4.0	12 430	.86	4250	5650	41 000	3.77	2.56
6	4.44	2.91	4.5	11 280	.91	4620	6250	41 900	4.17	2.62

[a] The depth of tensile steel given assumes that the reinforcement remained as originally placed in the form, and that the form bulged equally in both directions.

[b] In determining the ratio of the compressive stress at maximum load to the working stress the value of f_c used was computed by the straight-line formula, taking $n=10$ and $k=0.6$. The working stress was 1500 pounds per square inch. The top layer of bars was not counted as reinforcement as it came practically at the neutral axis.

[c] The ratio of the tensile stress at maximum load to the working stress is the factor of safety against failure by tension in the reinforcement.

The computed compressive stress in the concrete of the shallow beams at the maximum load was greater than the strength of the control cylinders, even though the ultimate strength of the concrete in the shallow beams had not been reached. The average strength of the control cylinders was 3670 pounds per square inch. The average computed compressive stress developed in the beams at the maximum load was 4400 pounds per square inch by the parabolic formula and 6075 pounds per square inch by the straight-line formula. The ratios of the computed flexural stress at maximum load to the cylinder strength were 1.20 and 1.66 for calculations by the parabolic and the straight-line for-

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mulas, respectively. These ratios are consistent with results found in previous tests.[1]

Under the caption "Ratio computed stress at maximum load to working stress" the values corresponding to the tensile stresses are the factors of safety of the beams. The values corresponding to the compressive stresses are not factors of safety, because the beams did not fail by compression. It is apparent that the factors of safety against compression failure must have been higher than the ratios under the caption "Compression."

3. SUMMARY

(*a*) The test indicated that with the gravel aggregate used it was practicable to obtain a smooth dense surface, such as is required in the shells of concrete ships, in spite of the narrow and constricted space between forms and the obstruction of the reinforcement.

(*b*) In the pouring test the maximum compressive stress developed in the concrete in flexure gave computed values averaging 66 per cent higher than the ultimate compressive strength of the concrete as determined by tests of control cylinders.

(*c*) The quality of concrete obtained by pouring into forms which were well pounded during the pouring was quite uniform; the maximum loads and computed stresses obtained in flexure tests of shallow beams made from the pouring test specimens were reasonably consistent.

(*d*) No variation in the strength of the beams due to differences in position in the pouring test specimen was determinable on account of the fact that none of the failures occurred by compression.

(*e*) The factors of safety against compression failure are not known, as the beams failed by tension. It is apparent, however, that if sufficient reinforcement had been provided to prevent tension failure, the factors of safety against compression failure would have been found to be greater than the values in Table 2 given as the ratio of the stress computed by the straight-line formula for the maximum load to the working stress.

(*f*) The working stress used in concrete ship design, 1500 pounds per square inch in flexural compression, provides an ample factor of safety against compression failure with concrete of the quailty used in these tests.

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[1] Slater and Zipprodt, Compressive Strength of Concrete in Flexure, Proc. Amer. Concrete Inst.; 1920.

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concrete and to set free the air bubbles. When the forms were removed it was found that a smooth job of concreting had been secured in all except one or two very small areas.

3. RESULTS

The results of this test are shown in Fig. 4. On these diagrams the ordinates represent the head of concrete on the pressure cell and the measured pressure of the concrete against the form. The abscissas represent the time in minutes elapsing after the surface of the concrete was at the level of the pressure cell. In every case the first reading on the cell was taken from 5 to 10 minutes after the concrete had come into contact with the cell.

From the curve shown for cell No. 1, it will be noted that as the head of concrete increased, the pressure also increased almost uniformly up to the maximum observed pressure. However, 30 minutes after the initial reading had been taken, or approximately 35 to 40 minutes after the concrete had reached the level of the cell, the pressure began to decrease in spite of the increasing head of concrete on the cell. Apparently this decrease was because the concrete which was in contact with the cell had begun to harden, causing it to act more in the nature of a solid than of a liquid. Cell No. 1 registered zero pressure about 150 minutes after the initial reading.

The maximum pressure indicated by cell No. 1 was 3.1 pounds per square inch; the head of concrete at that time was about 3.6 feet. Assuming that concrete weighs 150 pounds per cubic foot, and also that it acts as a liquid, its lateral pressure would be 3.6 times 150, or 540 pounds per square foot. As the measured pressure was equivalent only to 446 pounds per square foot, it indicates that the concrete did not act as a perfect liquid. A pressure of 446 pounds per square foot would be produced by a liquid whose weight was only 124 pounds per cubic foot.

In the diagram for cell No. 2 it will be noted that after the first foot of concrete had been poured above the cell level, very little more was placed for about 32 minutes, during which time the original batch had begun to harden and the pressure decreased slightly. After a further interval of 30 minutes an additional 2 feet of concrete was placed in the form, but the pressure on cell No. 2 did not change. Apparently this was due to the fact that the concrete in contact with the cell had already hardened. Zero pressure was registered by this cell at the end of 150 minutes, notwithstanding the gradually increasing head of concrete.

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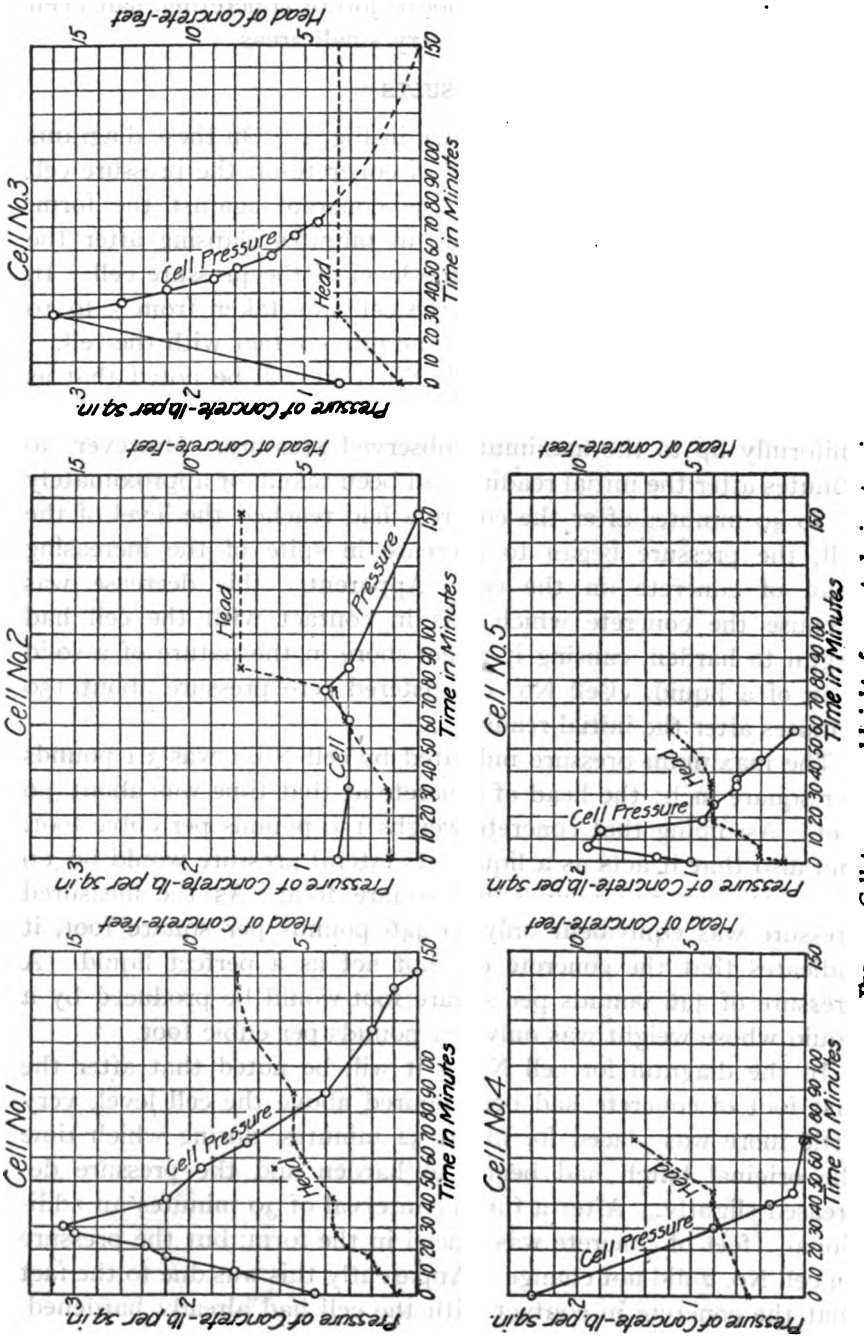

FIG. 4.—*Cell pressure and height of concrete during pouring*

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Cell No. 3 registered a maximum pressure of 3.2 pounds per square inch at the end of 30 minutes under a head of 3.6 feet. No more concrete could be added, as the form was full, but additional pressure readings were taken at intervals during the next 60 minutes. Each succeeding reading became lower as the concrete hardened. The curve is shown dotted after the last reading to indicate the probable rate of decrease to zero pressure. Apparently zero pressure would be indicated in about the same length of time as for cells Nos. 1 and 2.

Cells Nos. 4 and 5, which registered upward vertical pressures, present somewhat the same characteristics in the time-pressure curves as the cells measuring lateral pressure, except that they show smaller pressures and reach their maxima sooner. They also registered zero pressure after a shorter time interval.

Only two readings were taken on cell No. 6, and these have not been plotted. Cell No. 7 showed no indication of pressure whatever, and it was found after removal of the forms that no concrete came into contact with the cell.

4. SUMMARY

(*a*) The maximum pressure against the forms during pouring of concrete was found to be equivalent to that of a liquid weighing about 124 pounds per cubic foot.

(*b*) The maximum pressure was found to be that due to the head of concrete existing at the end of about 40 minutes. After that time the pressure gradually decreased in spite of increasing head of concrete.

These tests suggest the desirability of conducting further tests to determine (1) the relation between the pressure on the forms and the rate of increase in the head of the concrete, and (2) the influence of hardening of the concrete upon the pressure under increasing head.

WASHINGTON, June 15, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 176

SLUSHING OILS

BY

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Bureau of Standards

OCTOBER 14, 1920

PRICE, 5 CENTS

Sold only by the Superintendent of Documents, Government Printing Office
Washington, D. C.

WASHINGTON
GOVERNMENT PRINTING OFFICE
1920

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SLUSHING OILS

By Percy H. Walker and Lawrence L. Steele

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I. INTRODUCTION

Recently the Bureau of Standards has had numerous inquiries regarding information as to methods of protecting from corrosion metal in storage for rather long periods. These requests generally refer to the protection of metal articles which are of such a nature as not to allow the application of metallic coatings, such as galvanizing, or of vitreous enamels, paints, or varnishes. It is obvious that with bearing parts of machinery, coatings such as the above, which form dry and hard films and can not be easily removed, are not suitable. Our attention is therefore confined to a consideration of protective coatings which remain in a soft condition so that they can be easily removed at any time.

Regarding the length of time these coatings will afford adequate protection, there is little information available. It is well known that good oil paint will protect steel for several years when exposed to the weather, and that a good varnish will protect steel for several months. The protection afforded by paint and varnish to articles when kept indoors is very much greater than when exposed to the rays of the sun. While no data based on experiments are available, it is safe to assume that coating with a good grade of varnish and storing protected from the weather may be expected to preserve metal for several years. Paint under similar conditions may be expected to preserve metal for a considerable lnumber of years. As mentioned above, however, these two types

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of materials are not applicable for use on the bearing parts of machinery, or on many metallic articles which must be protected from corrosion. Such bright metal is protected by coating it with a layer of nondrying grease, usually known as slushing oil or slushing compound.

II. NATURE OF SLUSHING OILS

Slushing oils or slushing compounds are of a varied nature, sometimes being straight mineral greases (petroleum residuum, such as petrolatum), sometimes mixtures of mineral and animal greases with or without the addition of soap (such as lime rosin or lime fatty acid soaps), finely divided mineral matter, and volatile solvents. Sometimes the petroleum base is mixed with rosin or rosin oils. For example, certain branches of the Army used for a number of years on rifles and guns a compound composed of petrolatum mixed with not over 20 per cent rosin. Mineral oils as fluid as light spindle oils have been used for this purpose, but from the discussion which will follow it will be seen that such thin products are of very doubtful value except for very limited and particular purposes.

From one of the primary requirements of such materials, that they should be easily removed when desired, it follows that it is impossible to get a material of this nature which can not be rubbed off, although there is a very marked difference in ease of removal by abrasion.

III. DESIRED PROPERTIES OF SLUSHING OILS

An ideal slushing compound is one which can be easily applied to all kinds of metal surfaces by a variety of methods. It should coat the surface with a sufficiently thick and impervious film to exclude moisture and air (to prevent rusting), should remain in position for an indefinite length of time and yet be completely removable from the surface without undue labor. The material should itself have no corrosive action on any kind of metal. It is believed that the above statements give the essential requirements of a slushing compound. In addition, however, it is very desirable that the coating formed should be sufficiently transparent so that inspection to detect beginning of corrosion can be made without cleaning the article.

Metal articles which are absolutely protected from moisture, air, and other gases will never corrode. It therefore follows that any coating which will permanently exclude moisture, air, and

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any other gases commonly met with in the atmosphere, and which itself has no corrosive action must of necessity protect the metal from rust. There is, of course, no material which will to the fullest extent meet all of the above requirements, but for practical purposes it is believed that a number of materials of the varied composition indicated above answer very well for this purpose. No such materials in very thin films are absolutely impervious to air and moisture, hence very fluid compounds that leave only very thin films on the metal will not give as good protection as the same character of compound of a thicker consistency.

IV. OBJECTION TO HARD AND FAST REQUIREMENTS AS TO COMPOSITION OF SLUSHING COMPOUNDS

It follows from what has been previously said regarding variation in the composition of slushing compounds that any definite limitation on composition of the material desired may exclude from consideration perfectly satisfactory materials which do not meet the particular requirements as to composition and will thus tend to increase price.

In some cases we have noticed composition requirements which not only limit in an unjustifiable manner the choice of material for making a satisfactory article, but in some cases are so worded that it is practically impossible to secure any material which will meet the requirements. For example, we have seen specifications which called for practically pure petroleum products free from even a minute trace of acid. This clause was probably introduced because the acids most commonly thought of, mineral acids and the strong organic acids, are corrosive to metal, and such substances should not be present in any material intended for protecting metal from corrosion. On the other hand, some resin acids have practically no effect on the common metals, and resins containing these acids are frequently very valuable additions to slushing oils, producing products which have superior physical properties to the original petroleum base. Even without the addition of such resins, the petroleum base will never be free from small quantities of acid. It is well known that all petroleum products when exposed to light and air develop more than traces of acid (Waters, B. S. Scientific Papers, No. 153, and Technologic Paper No. 73).

Another requirement which has appeared in certain specifications and which appears to have originated in erroneous application of certain well-known facts is that the material should con-

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tain chemicals that are known to be rust inhibitors. It would, undoubtedly, be a good thing to secure a slushing oil to which actual rust-inhibiting properties had been given. Unfortunately, there is no known method of so altering these materials as to render them anything but at best inert in themselves. Regarding the inhibition of rust, it is of course well known that certain chemicals under particular conditions produce a so-called passive condition of iron, leaving the surface in a condition which resists rust for limited periods of time or resists rust under particular conditions of exposure. The well-known example of treating iron with very concentrated nitric acid might be mentioned, and of course it is ridiculous to consider that nitric acid, because under certain conditions it renders iron passive, would be a good material to add to a material for protecting iron from rust.

Iron and steel immersed in water solutions of strong alkalies will not rust as rapidly as when immersed in pure water. The water solutions of soluble chromates are among the best known rust-inhibiting substances, and if iron or steel is kept immersed in an aqueous solution of soluble chromate, it will resist rust for a much longer time than the iron immersed in water. This property of soluble chromates appears to have been the basis of the idea that rust-resisting chemicals could be satisfactorily added to slushing oils. One can not predict the action of chemicals when incorporated in any way with an oily medium from the action of these same chemicals when in water solution, as is shown very clearly by some very interesting experiments which were conducted on paints.

It had been found that some paint pigments, when suspended in water, had a marked influence on the speed with which iron and steel immersed in the water rusted. A large number of paint pigments were tested in this manner, and it was found that they could be divided into three classes—those in which the iron or steel rusted very much slower than in pure water, which for sake of convenience were called water inhibitors; those in which the iron or steel immersed in water containing pigment rusted in about the same degree as in pure water, which were called indeterminates; and those in which the iron or steel immersed in water containing the pigment rusted distinctly more rapidly than when in pure water, which were called water stimulators. This fact attracted so much attention that very extensive tests were made to ascertain whether these same pigments in an oily vehicle would behave in the same manner as they did in suspension in water. These

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tests were carried out under the direction of two committees of the American Society for Testing Materials.[1] The committees of this society carefully inspected these tests, which extended over a period of more than five years, and in rendering their final report made the following statement:

The summarized results indicate that the water test as at present developed can not be depended upon to give accurate information as to the protection wliich pigments will or will not afford metal structures when such pigments are applied in oil media. The summarized results also indicate that the arbitrary designations, inhibitive, indeterminate, and stimulative, as referring to the results of the water test, are not always applicable to the same pigments when used with linseed oil for protective coatings.

A single illustration taken from tests on numerous pigments by the above-mentioned committee will convince one that this conclusion was entirely justified as respects paints. Among the pigments selected were two samples of Prussian blue, Nos. 44 and 45. No. 44 Prussian blue was intentionally selected as one which in the preliminary water test acted as a stimulator of corrosion. No. 45 was one which under the water test acted as an inhibitor of corrosion. This Prussian blue No. 45 contained some chromate. In June, 1913, before the conclusion of this test, it was necessary to remove some of the panels, and the committee selected for discontinuing a number of panels which at that time were considered as having failed; among them was the inhibitive Prussian blue No. 45, whereas the stimulative Prussian blue No. 44 was not included in this list. It is probable that these observations were never carefully considered by the people who have advocated the use of so-called rust-inhibiting chemicals in slushing oils. So far as the writers know, all manufacturers who have attempted to produce slushing oils containing rust-inhibitive chemicals have prepared a product which was essentially an emulsion of a water solution of sodium or potassium chromate in a straight petroleum residue base. A careful consideration of the nature of this mixture would lead one to the conclusion that the water solution of the chromate could not possibly have any beneficial effect. Such a material when completed would consist of a large quantity of heavy petroleum product with minute globules of water solution of chromate distributed through it. These globules do not come in contact with the metal on which the slushing compound is coated and hence can have no effect as rust inhibitors. Soluble chromates

[1] A. S. T. M. Proceedings, IX, p. 106, 1909; X, p. 75, 1910; XI, p. 192, 1911; XIII, p. 369, 1913; XIV, Part 1, p. 259, 1914; XV, Part 1, p. 228, 1915.

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frequently cause serious injury to the skin, and some users of slushing oils containing chromates have been obliged to abandon them on this account.

V. LABORATORY TESTS ON SLUSHING COMPOUNDS

A few months after the United States entered the Great War, the Bureau of Standards started to receive samples of rust preventives from branches of the War Department. This material was intended for use on guns, rifles, and small arms and was purchased on a somewhat indefinite specification. At the request of the War Department the following tests were made on these samples: (1) Test for mineral acid; (2) test for rosin; and (3) an adhesion test.

A test was made for mineral acid by heating some of the sample with distilled water and qualitatively testing the water portion for acid in the usual way. Rosin was detected by the Liebermann-Storch reaction. The adhesion test was performed as follows: The sample was smeared on both sides of a tared copper plate 2 inches square and 0.0063 inch in thickness. This plate was then suspended by one corner in an oven and maintained at a temperature of 65° C (149° F) for 30 minutes. After cooling, the plate was weighed. An increase in weight of the plate of not less than 65 and not more than 85 mg was required.

The above tests probably gave very little information as to the merits of a given slushing oil. From results of work done later, it is believed that no adhesion test, where the film of oil remaining adherent to a test plate under set conditions is weighed, can be relied upon to differentiate between good and bad rust preventives.

Practically all the samples of slushing oil examined for the War Department by the methods outlined above were straight mineral oils of the consistency of petrolatum and did not contain thinners or mineral matter.

During the summer of 1918 the Bureau of Standards received at different times various samples of rust preventives from the War Department with a request in each case that certain extravagant rust-inhibiting qualities claimed by the manufacturer be investigated.

These compounds were analyzed in so far as it was possible and in addition it was thought best that some comparative test be made, using one of the regular samples of rust preventive in use by the War Department as a standard of comparison. To

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this end exposure tests on bright steel plates were made. These exposure tests have proved to be of very great value in the evaluation of rust preventives, and the details of the method will be explained at this point: Bright cold-rolled steel of No. 26 gage was cut into pieces measuring 4 by 6 inches and carefully cleaned by the use of cotton waste and benzene. The sample of rust preventive was then applied to these plates either by dipping, brushing, or smearing on with a spatula, according to the nature of the material. It was found convenient to tack the plate to a board before applying the slushing oil, so that the coated plate could be carried without danger of disturbing the film of rust preventive. The coated plates were allowed to stand for one day in the laboratory in an upright position and were then exposed outdoors at an angle of 45°, facing south. They were inspected at intervals of every few days and a record kept of the condition of the various plates.

These exposure tests proved to be of great value in the comparison of a number of samples. The length of time before rust appeared on the plate was influenced very largely by the kind of weather and the season of the year, so that the tests were only comparative. These first exposure tests indicated one fact which has been confirmed in nearly all cases by later series of tests, namely, oils of thin consistency give very poor protection to steel exposed to weather as compared with oils of greater body and hence greater adhesiveness. This is a conclusion which seems most logical.

At the time when these first exposure tests were made, an interesting phenomenon was noted. A compound was being examined, the manufacturers of which claimed it possessed rust-inhibiting qualities. This material was found to be essentially an emulsion of a small amount of a water solution of a chromate in heavy mineral oil The rust-inhibiting properties of chromates in slushing oils have already been discussed on page 6. When this sample was applied in the usual way to bright steel plates, either by smearing on the compound from the original container or else by applying the melted material, and the plate allowed to stand in an upright position in the laboratory, it was noted that the film of rust preventive slipped down the surface of the metal. The material did not melt on the panel and run off, but the whole film moved down the plate like a glacier and left the surface of the metal bare. At first it was believed that the slipping of this

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particular rust preventive was due to the small amount of water present in the oil chromate solution emulsion. However, other slushing oils were found later which were free from water and which showed the same phenomenon of slipping from bright metal surfaces.

The cause for this slipping observed in cases of certain rust preventives has not been determined. It is readily seen that a property of slipping from bright metal is a very serious defect in a slushing oil. At the suggestion of the Bureau of Standards, certain Government specifications have included the following clause or a similar requirement:

> When a highly polished steel plate 3 by 5 inches, coated with rust preventive, is allowed to stand on the shortest side, with the longest side vertical, for one week, there shall be no slipping of the rust preventive and no portion of the steel surface shall be exposed through movement of the protective coating.

The above clause, it was believed, would eliminate those rust preventives which possessed the serious defect of slipping.

After the signing of the armistice different branches of the War Department became concerned in the problem of storage of guns, airplane engines, machines, etc. The Bureau of Standards received numerous requests for information as to the best methods for the protection of metal from corrosion during storage for long periods of time. Mimeographed sheets entitled "Notes on Protection of Metal in Storage" were prepared and issued on January 4, 1919. In these notes tentative specifications were outlined for three different types of rust preventive: Brushing consistency, dipping consistency, and an opaque compound for special uses.

These specifications did not dwell upon composition of material, but an attempt was made to include certain laboratory tests which, in conjunction with the comparative exposure tests, should give a basis for the evaluation of different samples of slushing oils.

A new adhesion test was prescribed which differed considerably from the War Department method previously described on page 8. This modified adhesion test was as follows:

> (a) Two polished copper plates 5 by 5 cm and not more than 1.0 mm thick, will be weighed and then coated on both sides with the slushing oil. One of these plates will be suspended by one corner for 30 minutes, in an oven at 75° C, the other in an oven at 65° C. The two plates will then be weighed. The oil remaining on the plate heated at 65° C shall weigh not less than 60 mg and the oil remaining on the plate heated to 75° C shall not be less than three-fourths as much as that remaining on the plate heated to 65° C.

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(b) When applied to a polished metal plate 10 by 15 cm and the plate then held in a vertical position at any temperature between 15° and 35° C for 5 days, the coat of slushing oil shall remain adherent to the metal, shall not slide appreciably from the top edge of the plate nor leave any portion of the surface.

The clause which required that 75 per cent as much material remain on the plate at 75° C as remained at 65° C was designed to eliminate those rust preventives which rapidly lose their adhesive qualities with a slight increase in temperature. The weight of material required to remain on the plate after 30 minutes at 65° C was arbitrarily taken at 60 mg or more in the case of slushing oils of brushing consistency and 30 mg or more in the case of slushing oils of dipping consistency.

Experience with these adhesion tests has shown them to be of doubtful value in the examination of slushing oils. This point will be discussed in full further on.

In the question of acid content of rust preventives a very radical departure was made. Most slushing oil specifications required that "no acid be present." As already pointed out on page 5, pure mineral oils are likely to develop traces of acid on exposure to light and air. Furthermore, the detection of traces of acid in a petroleum is not certain by present methods. For these reasons it was not specified that no acid be present, but a requirement was added in the specification that "the rust preventive in question should not corrode bright copper, brass, nickel, or iron after five days." It is firmly believed that this requirement in regard to corrosion effectively excludes any slushing oils which contain harmful acid in quantity sufficient to cause trouble in actual use. Harmful acids which may be present in rust preventives cause stains to appear upon bright copper and brass in a comparatively short time, and therefore these metals act as sensitive indicators of the presence of corrosive substances in the slushing oil.

The exposure tests made on bright steel and already described in some detail are believed to be the most important of all the tests proposed in the new tentative specifications for slushing oil.

Since almost any rust preventive in use may be subject to exposure to the weather, the most vital property of a good slushing oil should be that it give an impervious and lasting film on metal, even when subjected to the action of rain, snow, and rapidly changing temperature. The great objection to exposure tests is the length of time that is needed to make them properly. Plates must be exposed for at least 30 days, and better for a space

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of two months, in order to draw any valuable conclusions. The time of the year must also be taken into account, because hot summer weather with rain is much more severe on rust preventives than cold winter weather with snow and ice.

A few months after the circular "Notes on Protection of Metal in Storage" had been issued, one of the purchasing bureaus of the War Department sent in a large number of samples of slushing oil with a request that an examination of these materials be made to determine if possible which brands were best suited for rust prevention. There were 44 samples in all, and tests were begun at once, using the methods of test outlined in the specifications of the circular of January, 1919.

The 44 samples were first grouped into three classes: Class 1, the transparent materials of brushing consistency at temperatures above 10° C (50° F); class 2, the opaque materials of brushing consistency at temperatures above 10° C (50° F); class 3, the materials of dipping consistency at temperatures above 10° C (50° F).

Each one of the 44 samples was examined under all the laboratory tests given in the circular, and in addition each material was ashed and the percentage and nature of the ash determined, the acid number was determined, and a qualitative test for rosin was made in each case.

The results of the laboratory tests will be found in Table 1.

Exposure tests lasting for a period of 60 days were made on all the samples. The materials which flowed readily at room temperature were applied by flowing an excess of oil over the plate and allowing the surplus to run off. The materials which were semisolid at room temperature were applied in two ways: (1) One set of plates was coated by smearing on an excess of material as it came from the container; (2) another set of plates was coated with melted material. The plates were coated on July 18, 1919, and exposed to the weather on July 19, 1919. The weather conditions during the following 60 days were unusually severe. There were many hard rains with intervals of hot sunny days. Some of the materials which did not stand up well in the exposure tests might give good protection under less severe weather conditions, but the tests were intended to be severe. It is believed that a satisfactory rust preventive should stand up well on metal for at

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least 60 days when exposed to the most severe weather conditions which may prevail at points where the material is to be put in service.

The results of the exposure tests upon the 44 samples after 60 days on the roof are given in Table 2.

In order to compare the results obtained in the exposure tests with the results in the laboratory tests, Table 3 was prepared. It must be kept in mind that several of the rust preventives of semisolid consistency were applied to the steel plates in the exposure test both as a paste and in a melted condition. Where different protection was given, depending upon the mode of application, that sample of slushing oil has been counted twice and the same compound may appear, for example, listed as both "fair" and "failure."

It is a fact worthy of note that nearly all the compounds of semisolid consistency gave the best protection in the exposure test when they were applied to the steel plate in a melted condition.

An examination of Table 3 shows that a strict interpretation of the laboratory tests given in the tentative specification of January, 1919, would have excluded many compounds which showed up well on exposure and would have passed at least three compounds which failed in the exposure test. For example, if the five compounds excluded in the laboratory tests because the resulting film was not removable with kerosene are not considered, we find that a strict interpretation of the adhesion and slipping tests alone would have excluded one compound rated as perfect in the exposure test, one compound rated as good, and eight compounds rated as fair, while two samples rated as complete failure and one sample rated as partial failure would have been accepted from the results of the laboratory tests alone.

The above comparison shows clearly that it was unwise to depend entirely upon laboratory tests outlined in the tentative specifications for decision in regard to acceptance or rejection of rust preventives.

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TABLE 1.—Laboratory Tests on 44 Samples of Slushing Oils

B. S. No.	Slipping	Corrosion			Adhesion			Ash		Acid No.	Rosin	Film
		Steel	Copper	Brass	At 65° C	At 75° C	Per cent	Per cent	Contains			
					Mg	Mg						
55784	O. K.	O. K.	O. K.	O. K.	57.6	34.2	59.5	0.07	Fe	27.7	Not detected	
55785	Slipped slightly on steel; did not leave surface.	do	do	do	91.0	71.2	78.4	.07	do	.12	do	Black opaque.
55786	O. K.	do	Corroded	Corroded	50.1	46.3	92.4	.06	do	41.9	Present	
55787	Slipped badly	do	O. K.	O. K.	83.3	64.3	77.2	.54	Na, Cr	.19	Not detected	
55788	do	do	do	do	60.1	59.7	99.4	.34	do	.19	do	
55789	do	do	do	do	83.6	45.4	54.2	.70	do	.12	do	
55790	O. K.	do	do	do	50.3	31.1	61.8	.00		(a)	do	
55791	Slipped badly	do	do	do	71.8	52.4	73.0	.01		.06	do	
55792	do	do	do	do	105.3	64.3	61.0	.67	Ca	.12	do	
55793	Slipped	do	do	do	76.2	39.6	52.0	.04		.25	do	
55794	O. K.	do	do	do	268.0	264.4	98.6	.22		.37	do	
55795	do	do	do	do	28.0	27.6	98.5	.00		.12	do	
55796	do	do	Slightly corroded	Slightly corroded	134.4	97.0	72.0	.20		.19	do	Opaque.
55797	do	Rusted under film	O. K.	O. K.	82.8	27.2	84.5	.00		.03	do	
55798	do	O. K.	do	do	678.6	668.6	98.5	.50		1.99	do	Black opaque.
55799	do	do	do	Corroded	30.4	30.0	98.7	.50	Mn, Ca	7.2	Present	
55800	do	do	do	O. K.	146.2	110.8	75.6	.60	do	5.4	do	
55801	do	do	do	do	2,025.0	1,979.0	97.6	2.85	Fe, Ca	.31	Not detected	Do.
55802	do	do	do	do	106.2	85.2	80.4	24.0	Ba, Zn, S, SO_4	.19	do	White opaque.
55803	do	do	do	do	101.0	83.2	82.3	7.00	Pb	.25	do	Opaque.
55804	Slipped	do	do	do	46.8	46.0	98.3	.03		.12	do	
55805	O. K.	do	do	do	348.2	294.2	84.5	18.00	Pb, Fe	.68	do	Dark brown opaque.
55806	do	do	do	do	38.0	37.8	99.5	.24	Mn, Pb, Ca	18.6	Present	
55807	Slipped badly	do	do	do	84.0	65.8	78.3	.00		.12	Not detected	
55808	O. K.	do	do	do	89.4	71.4	79.8	.00		.19	do	
55809	do	do	do	do	67.0	48.0	71.6	.00		.03	do	
55810	Slipped badly	do	do	do	44.4	43.4	97.7	.01		.03	do	
55811	O. K.	do	do	do	47.2	47.0	99.5	.00		.03	do	
55812	do	do	do	do	52.8	51.8	98.2	.00		.37	do	
55813	do	do	Corroded badly	do	45.4	45.2	99.5	.00		22.60	do	

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55814	do	do	O. K.	do	50.8	49.2	96.8	.06			3.50	Present		
55815	do	do	do	do	56.2	57.4	66.5					Not detected		
55816	Slipped badly	do	do	do	72.8	57.8	73.4	.00			.03	do		
55817	Slipped slightly	do	do	do	70.4	70.4	100.0	.00			.00	do		
55818	Slipped badly	do	do	do	75.5	97.0	75.5	.00			(a)	do		
55819	O. K.	do	do	do	47.6	48.2	101.3	1.30	Mn,Pb,Ca		9.27	Present	Set to teeth.	
55820	do	do	do	do	138.2	125.2	97.8	.00			16.2	do	Do.	
55821	do	Rusted under film	do	do	36.6	85.4	96.8	.00			.03	Not detected		
55822	do	Rusted slightly under film.	Slightly corroded	do	40.0	88.2	95.5	.00			.03	do		
55823	do	do	do	Corroded	121.0	85.5	70.6	.00			12.6	do		
55824	do	O. K.	Corroded badly	Corroded badly	181.2	144.4	79.7	.00			18.0	do		
55825	do	do	do	do	126.8	87.8	69.4	.00			16.4	do		
55826	do	do	do	do	125.0	104.8	83.8	.01			22.1	do		
55827	do	do	O. K.	O. K.	118.2	91.6	80.0	.06			.50	do	Opaque.	

a Slightly alkaline.

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TABLE 2.—Results of Exposure Tests for 60 Days upon 44 Samples of Rust Preventive Received from the War Department on July 11, 1919

B. S. laboratory No.	Materials [a]	Appearance of first rust	Condition of plate after 60 days [b]
55784	L	After 2 days	Complete failure.
55785	L	After 35 days	Partial failure.
55786	L	After 2 days	Complete failure.
55787	S	do	Fair.
55787	M	After 22 days	Do.
55788	L	After 4 days	Complete failure.
55789	S	After 2 days	Partial failure.
55789	M	After 4 days	Fair.
55790	L	After 2 days	Complete failure.
55791	S	After 3 days	Partial failure.
55791	M	After 60 days	Good.
55792	S	After 46 days	Fair.
55792	M	After 27 days	Partial failure.
55793	S	After 46 days	Fair.
55793	M	After 26 days	Do.
55794	L	After 60 days	Good.[c]
55795	L	After 2 days	Complete failure.
55796	L	do	Do.
55797	L	do	Do.
55798	L	None	Perfect.[c]
55799	L	After 4 days	Complete failure.
55800	L	After 60 days	Good.
55801	S	None	Perfect.
55801	M	After 32 days	Fair.
55802	S	None	Perfect.
55802	M	do	Do.
55803	S	After 2 days	Partial failure.
55803	M	do	Fair.
55804	S	do	Partial failure.
55804	M	do	Complete failure.
55805	S	None	Perfect.
55805	M	do	Do.
55806	L	After 18 days	Complete failure.
55807	S	After 2 days	Partial failure.
55807	M	After 46 days	Fair.
55808	L	After 10 days	Complete failure.
55809	S	After 14 days	Partial failure.
55809	M	After 2 days	Do.
55810	S	do	Do.
55810	M	do	Do.
55811	S	do	Fair.
55811	M	After 18 days	Do.
55812	L	After 2 days	Complete failure.
55813	L	do	Do.
55814	L	None	Perfect.
55815	S	After 2 days	Fair.
55815	M	do	Do.
55816	S	do	Complete failure.
55816	M	do	Do.
55817	S	After 10 days	Fair.

[a] Materials marked "L" were liquids at room temperature and were applied to the plates as such; materials marked "S" were semisolids and were smeared on the plates as a paste; in addition the semisolid materials were melted and applied to plates as liquids, and in such cases the mark "M" appears.

[b] Ratings are given as follows: "Perfect," where the exposed steel plate shows no evidence of rust; "good," where the plate shows evidence of only a superficial and slight amount of rust; "fair," where the plate shows decided evidence of rust, but where a large percentage of the surface is still free from rust; "partial failure," where at least one-third of the surface of the plate is covered with rust; "complete failure," where the entire surface of the plate is covered with rust.

[c] At the end of the 60 days' exposure this coating could not be readily removed by waste wet with kerosene.

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TABLE 2.—Results of Exposure Tests for 60 Days upon 44 Samples of Rust Preventive Received from the War Department on July 11, 1919—Continued

B. S. laboratory No.	Materials	Appearance of first rust	Condition of plate after 60 days
55817	M	After 14 days	Fair.
55818	S	After 2 days	Partial failure.
55818	M	do	Do.
55819	L	After 3 days	Complete failure.
55820	L	None	Perfect.[a]
55821	L	After 2 days	Complete failure.
55822	L	do	Do.
55823	L	After 22 days	Fair.[a]
55824	L	After 27 days	Do.[a]
55825	L	After 22 days	Do.[a]
55826	L	After 2 days	Complete failure.
55827	L	do	Do.

[a] At the end of the 60 days' exposure this coating could not be readily removed by waste wet with kerosene.

TABLE 3.—Relation Between Ratings of Exposure and Results of Laboratory Tests upon 44 Samples of Rust Preventive from the War Department

Rating in exposure	Compounds	Passed all laboratory tests	Failed adhesion test	Failed slipping test	Failed corrosion test	Failed both adhesion and slipping tests	Failed both adhesion and corrosion tests	Film not removable with kerosene
Perfect	6	3	1	0	0	0	0	2
Good	3	1	0	0	0	1	0	1
Fair	13	2	2	3	1	3	2	0
Partial failure	10	1	1	3	0	5	0	0
Complete failure	19	2	4	1	5	2	3	2

Exposure tests on steel panels seemed to be a satisfactory method for distinguishing good slushing oils from poor ones, but this method was open to at least two very serious objections. In many cases the decision as to acceptance or rejection of rust preventives must be made in less than 60 days, and in some cases it would not be possible to await the results of an exposure test for a period of even 30 days. The results of exposure tests on rust preventives are influenced to a very great extent by weather conditions. Hot summer weather is a much more severe test upon a slushing oil than cold winter weather, and it is conceivable that a compound might be accepted upon the basis of 60 days exposure in winter, while the same material might be rejected from the results of summer exposure.

It was therefore seen to be very important that some laboratory tests be devised which would tell in a short time whether a given rust preventive would protect exposed metal under any conditions of weather, especially the severe conditions met with

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during the summer near the seacoast with the accompanying salt-laden air. In some of the work already described it had been noticed that certain compounds seemed to be literally washed from the exposed steel plate by the first heavy rainfall. To distinguish this fatal defect in slushing oils, an artificial rain test was devised which will be described in detail further on.

It was considered that the failure of rust preventives which were not washed off the plate by rainfall was due to some one or more of three things: Either the material itself corroded metal, or it was more or less porous and allowed moisture to penetrate the film, or the rust preventive did not remain adherent to the metal owing to slippage or melting off of the coating by the heat of the sun.

It was believed that the detection of corrosive materials in the rust preventive itself was covered in an efficient manner by the corrosion test given in the original tentative specifications of January, 1919, and no change was made in this test.

A so-called oven salt-spray test was devised in which an attempt was made to simulate to a certain extent the combined conditions of hot weather and salt-laden air near the ocean. This new test was also designed to show up rust preventives which were porous or possessed the defect of slipping.

A specification for slushing oils was drawn up which embodied the new oven salt-spray test and the rain test. A comprehensive examination of a series of rust preventives was made according to these tests, and the specification was altered in certain details until the form given below was the final result.

VI. PROPOSED SPECIFICATIONS FOR SLUSHING GREASES SUITABLE FOR THE PROTECTION OF EXPOSED BRIGHT METAL

1. The material shall furnish a coating that shall firmly adhere to all metal surfaces at all temperatures at which they may be exposed under natural conditions.

2. The coating shall permanently remain in such a condition that it can be readily removed with cotton waste wet with kerosene.

3. CORROSION TEST.—When applied to polished iron, steel, brass, and copper surfaces and exposed at any temperature below 100° C (212° F) for a period of not less than five days, there shall be no stains on the metal or other evidence of corrosion due to the slushing oil.

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4. OVEN SALT-SPRAY RAIN TEST.—Four bright steel plates approximately 4 by 6 inches shall be tacked to separate boards and thoroughly cleaned by the use of benzene. The slushing oil to be examined shall be melted, if not already liquid, and an excess flowed over the plates. One of the plates shall be kept in a vertical position in the laboratory for 24 hours and then placed in a rack on the roof so that the plate shall be inclined at an angle of 45° to the vertical, facing south. The plate shall be sprayed lightly with a 3 per cent salt solution on the first day. The other 3 plates shall be hung in a vertical position in an oven and maintained at a temperature of 45 to 50° C (113 to 122° F) for at least 48 hours. The plates shall be removed from the oven, allowed to cool, and 1 plate exposed on the roof in the rack described above. Another plate shall be kept in a horizontal position and lightly sprayed with a 3 per cent salt solution once every day for a period of 5 days. The fourth plate shall be placed under an intermittent shower, a vigorous shower being applied for approximately 3 to 4 minutes, then no water for about the same length of time. The shower is to be formed by allowing water to siphon at intervals from a 5-gallon tank into a metal trough, the bottom of which has three or four parallel rows of small holes (about one-sixteenth inch in diameter). The water should fall about 2 feet from the trough to the plate. This test should be continued for not less than 5 hours, the plate being held in a position about 60° to the vertical immediately under the falling water. At the end of 5 hours of this intermittent showering, the plate shall be placed in a horizontal position and allowed to remain with any adhering water for at least 24 hours. No rust shall be in evidence on any of the 4 plates after 5 days.

When time permits, the plates on the roof should be exposed for a period of 60 days, and at the end of this time there should be no appreciable rust in evidence.

DISCUSSION.—It is believed that any material which passes all the tests and requirements given above will protect bright exposed metal for a period of at least 60 days in any weather commonly encountered and will probably continue to protect exposed metal for many months. It has been found that the compounds which pass the above tests are usually greases of rather high melting point and should be applied to metal while melted, either by spraying or by dipping. We have seen a few samples that are liquid at ordinary temperatures which fulfill the requirements.

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In order to gain some insight into the way in which the new laboratory tests on slushing oils would work out in routine examination of rust preventives, tests were made upon 23 of the 44 samples from the War Department previously referred to. Those compounds which had been found to corrode metal in the original tests were not selected. Compounds which had been found to give a varnish-like film were also left out. The results of exposure of each one of these 23 compounds during the most severe weather in Washington (July and August) was known. Each of these 23 compounds was carefully put through the oven salt-spray rain test outlined in the new proposed specification.

Seven of the 23 samples had been rated as "complete failure" in the 60-day exposure test. All 7 of these slushing oils failed conclusively in the oven salt-spray rain test. Seven of the 23 samples had been rated as "partial failure" in the exposure test. Six of these samples failed conclusively and 1 passed the oven salt-spray rain test. Four of the 23 samples had been given a rating of "fair" in the exposure test. All of these samples passed the oven salt-spray rain test. One of the 23 samples had been rated as "good" in the exposure test and it passed the oven salt-spray rain test. Four of the 23 samples had been rated as "perfect" in the exposure test. Three of these samples passed the oven salt-spray rain test, while the other sample showed slight evidence of rust on the edge of the plate.

From the above results it is seen that the laboratory tests in the new proposed specifications would exclude 13 out of 14 samples which had been rated as unsatisfactory in the exposure test. At the same time 1 sample out of 9 of the compounds rated as satisfactory in the exposure test would be excluded.

The results obtained above on the 23 samples by the laboratory tests in the new proposed specification are believed to average as well as most laboratory tests when they are compared with the results of actual service conditions or conditions similar to actual service.

Laboratory tests are almost never perfect, but it is believed that the tests outlined in the proposed specification are as good as any that can be given at the present time. Attention is called to the fact that even where it is possible to run exposure tests extending over considerable periods of time and some brand of material is found to give good service, rapid laboratory tests are still of great use in determining uniformity of quality of the product. It has often been noticed that different lots of the same

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brand of rust preventive received at different times show wide variation in rust-preventing qualities. A rapid laboratory test can be used to determine the quality and uniformity of each shipment of slushing oil.

VII. GENERAL COMPOSITION OF SLUSHING OILS

During this investigation comparatively little work was done upon the composition of slushing oils. It was felt that it would be unwise to attempt in any specification for these materials to limit the manufacturer in the raw materials that he should use, because performance tests were more important than composition requirements.

Only a brief discussion of the general composition of commercial slushing oils will be given here, together with a description of some experimental batches of slushing oil prepared during the course of this investigation.

An examination of Table 1, on page 14, where the results of chemical tests upon 44 commercial samples of rust preventive are given, will give considerable insight into the composition of these samples. The majority of these materials showed a low acid number, together with a low ash content. These compounds are probably straight petroleum products of varying consistency from thin oils of low viscosity to petrolatum-like materials and thick petroleum residues.

The compounds which yield a considerable amount of ash may contain mineral matter as a filler (for example, one slushing oil seemed to be a mixture of lithopone and a heavy petroleum residue, while another seemed to be mainly iron oxide with a petroleum product), or they may be petrolatum-like compounds emulsified with a little water soluble chromate solution, supposed to act as a rust inhibitor.

The presence of small amounts of manganese with or without lead and calcium would indicate that some of the compounds are varnishes or mixtures of a petroleum product with varnish.

Several of the materials with high acid number were found to be blown vegetable oils. There were several black opaque compounds, which consisted of soft asphalt material or hard asphalt thinned with some solvent.

Rosin was detected in several of the compounds. In some cases it was present as an ingredient of varnish, while in other cases the slushing oil seemed to be a mixture of a petroleum product with rosin.

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Among the 44 samples there was one, Bureau of Standards laboratory No. 55 814, which appeared to be a mixture of petrolatum and rosin, thinned with some kerosene. This compound was given a rating of perfect in the exposure test. A mixture based on the formula given below was made up in the laboratory in an attempt to duplicate the sample above mentioned.

20 g rosin, "H" grade,
100 g petrolatum (U. S. P.),
10 cc kerosene.

The rosin was melted and mixed with the previously heated petrolatum and the resulting material thoroughly stirred. After the mixture had partially cooled the kerosene was added. The presence of the rosin greatly increased the adhesive property of the mixture over that of the original petrolatum.

Bright steel plates were coated with this material in the usual way and exposed to the weather on September 9, 1919. On May 5, 1920, the plates were brought into the laboratory and cleaned off by means of cotton waste wet with kerosene. The film of rust preventive was readily removed and the surface of the steel underneath was found to be smooth and practically free from rust. The plates were nearly as bright as when they were coated and exposed eight months before.

The results of the above exposure test indicate that a good slushing oil can be made from ordinary petrolatum and rosin. Where a rust preventive might be exposed to extreme hot weather, such as a temperature of 120° F met with in the sun's direct rays, it was thought best to add something to the petrolatum-rosin mixture to raise its melting point.

The U. S. P. petrolatum used was smeared over the bulb of a thermometer and heated in an air bath. At 37° C. a drop of grease fell from the thermometer and this point was taken as the melting point of the material. Mixtures of rosin with petrolatum showed melting points little higher than the petrolatum alone. For example, 30 parts of rosin and 100 parts of petrolatum mixed together hot showed a melting point of 40° C by the above method. A wax, like carnauba, with a high melting point, was found to raise the melting point of petrolatum to a much more pronounced degree. For example, 40 parts of carnauba wax and 100 parts of petrolatum were mixed hot, and the melting point of the cold mixture found to be 79° C. In a similar way 40 parts of candelilla wax and 100 parts of petrolatum was found to melt at 59.6° C.

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Two per cent of carnauba wax was found to raise the melting point of petrolatum to 52.5° C, 15.5 degrees higher than petrolatum alone.

Two formulas for a slushing grease of semisolid consistency are given below:

FORMULA A.—3 parts candelilla wax, 6 parts rosin, grade "H," 50 parts petrolatum (U. S. P.).

FORMULA B.—2 parts carnauba wax, 5 parts rosin, grade "H," 50 parts petrolatum (U. S. P.).

The above materials should be heated together at about 125° C until all the ingredients are melted and the mixture then thoroughly stirred and allowed to cool.

Both of the above mixtures were found to pass all the requirements for a slushing oil given in the proposed specification on page 18. The above mixtures were also applied to bright steel plates by flowing an excess of the melted grease over the surface. These plates were exposed to the weather and were found to be free from rust after several weeks' time.

The mixtures described under formula A and formula B are not claimed to be the best rust preventives which can be made to fulfill the requirements of the proposed specification, but are cited merely as examples of easily made compounds which were found to prevent rust in a satisfactory manner.

There are many manufacturers who employ different raw materials and make rust preventives of equal or better quality.

WASHINGTON, June 16, 1920.

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No. 177

SULPHUR IN PETROLEUM OILS

BY

C. E. WATERS, Chemist
Bureau of Standards

OCTOBER 20, 1920

PRICE, 5 CENTS
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SULPHUR IN PETROLEUM OILS

By C. E. Waters

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I. INTRODUCTION

Crude petroleum, from whatever source, contains sulphur. In the early days of the industry in this country, little or no attention was paid to this. The Pennsylvania oils contained but small amounts of this element, and the lighter distillates held lower percentages of sulphur than the crudes from which they were obtained. If this had not been so, the petroleum oils would no doubt have come into popular favor much more slowly than they did. Not only do oils which contain much sulphur have a disagreeable odor, but the products of their combustion are irritating and corrosive.

When the Canadian oil fields, and soon afterward those in Ohio and the adjacent States, were tapped, the removal of the comparatively large amounts of sulphur they contained became at once a refinery problem of the first importance. It is an interesting coincidence that Frasch, who years later developed the process by which the immense deposits of sulphur in Louisiana and Texas are worked, devised the first commercially successful method of removing sulphur from petroleum.

II. ORIGIN OF SULPHUR IN PETROLEUM

Several different sources of the sulphur and sulphur compounds which occur in petroleum have been suggested.[1] If the theory that the oil is derived from decayed animal remains be accepted, no other source of the sulphur need be sought, because this element is an essential constituent of all proteids.

It has been suggested that gypsum and other sulphates may be reduced by the oil. The sulphides thus formed could, according to the conditions, yield hydrogen sulphide or free sulphur. A portion of the latter might be deposited in the rocks. Again, oil percolating through these rocks might take up sulphur which was deposited in earlier ages.

III. FORMS IN WHICH SULPHUR OCCURS

1. FREE SULPHUR

Free sulphur has been found in numerous Roumanian[2] oils, as well as in some from California[3] and Texas.[4]

[1] Steuart, J. Soc. Chem. Ind., 19, p. 989, 1900; and Naphta, 10, p. 97, 1902; Peckham, Proc. Amer. Phil. Soc., 37, p. 137, 1898; Engler, Fortschritte naturw. Forschung, 1, p. 897; Mabery, Petroleum, 1, p. 516; 2, p. 319; Höfer, Erdöl u. s. Verw., Ed. 2, pp. 82 and 141; Höfer, Sitzber. k. Akad. Wiss. Wien, Math.-Nat. Kl., III, Abt. 1.

Nearly all of the data on the occurrence of sulphur in petroleum, as well as many literature references, were obtained from Engler-Höfer, Das Erdöl, 1, pp. 463 to 479.

[2] Bourqoui, Chem. Rev. Fett-Harz-Ind., 8, p. 210, 1901; Kissling, Chem.-Ztg., 26, p. 492, 1902.

[3] Peckham, Proc. Amer. Phil. Soc., 36, p. 108; 1897.

[4] Richardson and Wallace, Eng. Min. J., 73, p. 352, 1902; and J. Soc. Chem. Ind., 21, p. 316, 1902; Thiele. Chem.-Ztg., 26, p. 896, 1902.

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2. HYDROGEN SULPHIDE

Hydrogen sulphide is produced by the decay of organic matter, particularly under anaerobic conditions, and it is a frequent constituent of volcanic gases and occurs dissolved in subterranean waters. It is not surprising, therefore, that it is found in crude petroleum from many sources.[5]

Petroleum which contains free sulphur or organic sulphur compounds gives off hydrogen sulphide when heated, sometimes even at moderate temperatures. This appears to be somewhat at variance with what seems to be a well-established fact that the higher-boiling distillates contain a higher percentage of combined sulphur than the more volatile ones.

3. ORGANIC SULPHUR COMPOUNDS

It is very difficult to separate from crude petroleum any of its constituent hydrocarbons in pure form. Because of the ease with which many of them break down when heated, it is even more difficult to isolate and identify individual sulphur compounds, or even classes of compounds. However, members of more than one class of compounds have been identified. The evidence is not always as convincing as one could wish, for some of the compounds have been detected in distillates or found in the residues from crude oil. The ease with which hydrogen sulphide is formed by heating the oil shows that changes must take place in the organic sulphur compounds to some extent. No doubt those who have done most work in this field realize most fully the difficulties and limitations.

(*a*) THIOPHENE and some of its homologs apparently occur in some crudes, and their presence in certain light distillates has been proved by means of the indophenine reaction.[6]

(*b*) THE THIOPHANES are an interesting class of compounds which were obtained from the acid tar of a Canadian petroleum.[7] Although little is known of their structure, the empirical formula, $C_nH_{2n}S$, and their chemical behavior seem to show that they contain a polymethylene ring. They react readily with bromine,

[5] Vender, Riv. scient. industr., **27**, p. 14, 1895; and Chem.-Ztg. Rep., **19**, p. 61, 1895; Nawratil, Dingl. pol. J., **246**, p. 423, 1882; Bolley, Ibid., **169**, p. 124, 1863; Mabery, Proc. Amer. Acad., **31**, pp. 17 and 43, 1894; Richardson, J. Frank. Inst., **162**, p. 115, 1906; Thiele, Chem.-Ztg., **25**, pp. 175 and 433, 1901; Richardson and Wallace, J. Soc. Chem. Ind., **20**, p. 690, 1901; Kast and Künkler, Dingl. pol. J., **278**, p. 38, 1890; Peckham, Proc. Amer. Phil. Soc., **36**, p. 108, 1897; Rakusin, Untersuchung des Erdöls, p. 102.

[6] Krämer, Verh. Vereins Beförd. Gewerbe-Fl., **1885**, p. 296; Meyer and Nahnsen, Ber. d. Chem Ges., **18**, p. 217, 1885; Edeleanu and Filiti, Bull. soc. chim., Ser. 3, **23**, p. 384, 1900; Charitschkoff, J. Russ. Phys.-Chem. Soc., **31**, p. 655, 1889, and Chem.-Ztg. Rep., **30**, p. 476, 1906; Girard, Petroleum, **2**, No. 3, 1906, and J. du Pétrole, **1906**, p. 69.

[7] Mabery and Quayle, Proc. Amer. Acad., **41**, p. 89, 1905; and Amer. Chem. J., **35**, p. 404, 1906; Mabery, J. Soc. Chem. Ind., **19**, p. 508, 1900.

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but are not unsaturated compounds, because the amount of bromine evolved as hydrobromic acid is equal to that held in combination. By oxidation with alkaline permanganate solution, the thiophanes are converted into the corresponding sulphones, $C_nH_{2n}SO_2$.

The eight thiophanes isolated ranged in composition from $C_7H_{14}S$ to $C_{18}H_{36}S$.

(*c*) ALKYL SULPHIDES, OR THIO ETHERS, $(C_nH_{2n+1})_2S$, have been reported in certain Ohio crudes. Ten members of the series, from methyl sulphide, $(CH_3)_2S$, to hexyl sulphide, $(C_6H_{13})_2S$, were isolated.[8]

Kast and Lagai,[9] who may not have had the same crudes as Mabery and Smith, failed to find any compounds of this series in Ohio oils. On the other hand, Charitschkoff [10] detected alkyl sulphides in naphtha from Grosny.

The alkyl sulphides yield crystalline addition products with mercuric chloride.

(*d*) ALKYL HYDROSULPHIDES, OR MERCAPTANS, apparently occur in the distillates from Baku oil.[11]

(*e*) CARBON BISULPHIDE has been detected in petroleum ether (boiling point 50 to 80°C), but not in kerosene.[12] Compounds of the general formula, $C(C_nH_{2n+2})S$, have been obtained from certain Canadian petroleums.[13] They are apparently alkyl derivatives of carbon bisulphide.

(*f*) SULPHONIC ACIDS, RSO_3H, are said to cause the turbidity which is sometimes observed when distillates are allowed to stand for a long time.[14] They can be removed by vigorous agitation with a solution of caustic alkali.

(*g*) ALKYL SULPHATES.—Perhaps most of the compounds commonly called sulphonic acids are in reality alkyl sulphates, $(C_nH_{2n+1})_2SO_4$, which are formed when the oils are treated with sulphuric acid during refining. The saturated hydrocarbons do not react with the acid, but the unsaturated ones can form these esters by direct addition. For instance,[15]

$$2C_nH_{2n} + H_2SO_4 = (C_nH_{2n+1})_2SO_4.$$

---

[8] Mabery and Smith, Ber. d. Chem. Ges., 22, p. 3303, 1889; and Amer. Chem. J., 13, p. 232, 1890.

[9] Dingl. pol. J., 284, p. 69, 1892; 292, p. 116, 1894.

[10] J. Russ. Phys.-Chem. Soc., 31, p. 655, 1899; Chem.-Ztg. Rep., 21, p. 203; Chem. Centrbl., 1899, II, p. 920.

[11] Kwjatkowsky, in Höfer, Erdöl u. s. Verw., 2d ed., p. 82.

[12] Hager, Dingl. pol. J., 188, p. 165; 1867.

[13] The reference given by Höfer could not be verified, or the statement traced to its source.

[14] Veith, Dingl. pol. J., 277, p. 567; 1890.

[15] Vohl, Dingl. pol. J., 216, p. 47, 1875; Heusler and Dennstedt, Z. angew. Chem., 17, p. 864, 1904; Heusler, Ber. d. Chem. Ges., 28, p. 498, 1895.

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The true sulphonic acids, which have their sulphur directly combined with carbon, can not be hydrolyzed by boiling with water or with dilute acids or alkalies. By careful fusion with caustic alkali the sulphonic group can be replaced by hydroxyl, as in one commercial process for the manufacture of phenol:

$$C_6H_5SO_3H + KOH = C_6H_5.OH + KHSO_3.$$

The alkyl esters, on the other hand, can be hydrolyzed with comparative ease by boiling with dilute acids or alkalies:

$$(C_nH_{2n+1})_2SO_4 + H_2O = 2C_nH_{2n+1}.OH + H_2SO_4.$$

When heated, these esters break down and give sulphur dioxide and carbon, among other products. These are formed even at the moderate temperatures reached in determining the Maumené number.[16]

IV. IDENTIFICATION AND SIGNIFICANCE OF SULPHUR COMPOUNDS

With the possible exception of the "sulphonic" acids, there seems to be no attempt made to determine the nature and amount of the sulphur compounds in routine laboratory practice. The difficulty, or even impossibility, rather than a lack of interest, is probably responsible for this. The oil chemist must feel that not only the total amount of sulphur, but also the nature of the compounds in which that element occurs, influence the behavior of oils. This is particularly true of lubricating oils which are subjected to service conditions favorable to deterioration. Heating in contact with air naturally comes first to mind because of the ease with which so many sulphur compounds are oxidized.

A conclusive decision as to the real influence of sulphur compounds upon the behavior of lubricating oils can be made only by comparing a sulphur-free oil with the same oil to which definite amounts of known sulphur compounds have been added. It does not appear probable that such tests can be made in the near future. However, it might be desirable to collect data on oils that have deteriorated badly in service in order to learn whether they contain higher percentages of sulphur than other oils which have stood up longer under the same conditions.

In this connection it may be pointed out that sulphur and sulphur dioxide cause an increase in the amount of asphaltic matter which is formed when oils are heated in the air.[17]

[16] See footnote 15, and also Waters, B. S. Tech. Papers, No. 73, p. 12; 1916.

[17] Holde and Eickmann, Mitth. kgl. Materialprüfungsamt, 1907, p. 145; and Z. angew. Chem., 20, pp. 1263 and 1923, 1907; Waters, B. S. Tech. Papers, No. 4, p. 9, 1911; and J. Ind. and Eng. Chem., 8, p. 814, 1911; Southcombe, J. Soc. Chem. Ind., 30, pp. 261–264, 1911.

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Work done recently at this Bureau shows that of four oils which had "carbonization values," or percentages of asphalt formed by oxidation, of 0.02 or less, none contained more than 0.08 per cent of sulphur. Of five oils which had carbonization values of 0.34 or more, all contained at least 0.29 per cent of sulphur. From these few figures it would appear that no oil which contains much sulphur will have a low carbonization value.

Transformer oils are regarded as undesirable if they contain more than a very small proportion of sulphur, because this element is believed to attack the wires and also to increase the tendency to form sludge.

In specifications for fuel oils the maximum permissible percentage of sulphur is generally stated, because of the corrosive effect of the products of its combustion. This is even more important for kerosene, because the oxides of sulphur are irritating and may bleach fabrics in the home, as well as corrode metal objects. The kerosene itself usually has a disagreeable odor when it contains more than a very low percentage of sulphur compounds.

Sometimes the presence of sulphur is desirable, as in flotation oils and some cutting oils. In the first case it is essential that the particles of ore cling to the oil drops in order to become separated from the gangue. The metallic sulphides more easily become oily than the other minerals. This oiling effect is helped by the presence of sulphur, perhaps by its action on particles or surface films of oxide which are thus changed to sulphide.

The principal reason for using a cutting oil is to lubricate the chips so that they can slide off the tool easily. It has been claimed that sulphur causes the oil to cling better to the metal by forming a film of sulphide. If this be true, the film is of extreme tenuity, because the metal shows little sign of tarnishing, as a rule.

V. DETECTION OF SULPHUR AND ITS COMPOUNDS

1. COPPER TEST

Free sulphur, hydrogen sulphide, and perhaps other sulphur compounds, in oils can be detected by the formation of a dark spot of copper sulphide when two or three drops of the oil are left on the polished metal for a sufficient length of time. The "copper dish" test for gasoline is based upon this reaction. It is generally believed that the degree of darkening depends upon the amount of sulphur present. This is not true. A topped Mexican crude, which contained 4.05 per cent of sulphur, and which gave off streams of hydrogen sulphide when heated in a test tube, did not perceptibly darken copper in 72 hours. Even

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when a strip of the metal was partly immersed in the oil and kept at 295 to 300° C for 2 hours, the lower portion was only slightly tarnished. The end of the strip which was not under the oil was, however, much blackened and covered with a varnish-like coating, which was probably formed by partial oxidation of the oil.

Although the copper test can not be relied upon to show the presence of all sulphur compounds in oils, it is a delicate means of detecting free sulphur and hydrogen sulphide. A series of solutions was made by dissolving recrystallized sulphur in a white, medicinal oil which did not darken copper and which was found to contain too little sulphur to detect by analysis. The strongest solution contained 0.100 per cent of free sulphur. By suitable dilution, samples containing down to 0.0001 per cent, or 1 : 1 000 000, of sulphur were prepared. On each piece of freshly sandpapered, washed and dried piece of copper was placed two or three drops of oil. A blank test was run to check the original oil as well as the air of the laboratory. With 0.10 to 0.050 per cent of sulphur the copper was blackened in a few minutes, with 0.033 to 0.017 per cent the copper was perceptibly darker in 30 minutes, and with 0.01 per cent inside of an hour. When left in contact with copper for 20 hours, oil containing as little as 0.0002 per cent of sulphur, or 2 : 1 000 000, caused a just perceptible darkening of the metal. At the final dilution no action could be detected.

Similar tests were made with oil into which hydrogen sulphide had been passed for a few minutes. This solution, which instantly darkened copper, contained 0.037 per cent of sulphur, equivalent to 0.039 per cent of hydrogen sulphide. A portion of this was diluted to 100 volumes and then three or four times to 10 volumes each. The results of the tests with hydrogen sulphide are given in Table 1.

TABLE 1.—Copper Test for Hydrogen Sulphide

Dilution	Sulphur	Parts per million	Odor	Action on copper in 20 hours
Volumes	Per cent			
100	0.00037	3.7	Strong	Slight.
1 000	.000037	.37	Positive	Doubtful.
10 000	.0000037	.037	Very faint	None.
100 000	.00000037	.0037	None	Do.

It is evident that the copper test is much less delicate than the sense of smell.

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2. SODIUM TEST

If the oil is not too volatile, but can be heated to 250 to 300° C without boiling out of the test tube, the sodium test can be applied. A test tube containing a few cubic centimeters of the oil is clamped in an inclined position over a flame which is regulated so that the oil is near its boiling point. A clean piece of sodium, 2 or 3 mm in diameter, is dropped in before the heating begins. After half an hour or longer, the oil is allowed to cool and is then poured off. The blackened lump of sodium is rinsed off with benzene, and dissolved in a few drops of water or 50 per cent alcohol. To the alkaline solution there are now added a few minute particles of sodium nitroprusside. If any sodium sulphide is present, the solution becomes pink to reddish purple, according to the amount. When only traces of sulphur are present, care must be taken not to mask the reaction by adding too much nitroprusside. Hydrogen sulphide does not produce a colored compound, so that the solution must not be acidified.

Attempts to utilize the sodium method for the determination of sulphur were unsuccessful, because duplicate results varied too widely.

VI. DETERMINATION OF SULPHUR

1. METHODS EMPLOYING OXIDIZING AGENTS.

(*a*) CARIUS METHOD.[18]—Although capable of giving highly accurate results, few chemists employ it when another can be used. It is not only troublesome and dangerous, but it has the disadvantage that only a few decigrams of material can be used in a determination. It is hardly applicable to oils low in sulphur. Even with oils that contain high percentages, there is danger of getting low results, owing to incomplete decomposition of the oil. Even when no more oily drops are to be seen, the oxidation may not be complete, as will be discussed more fully further on.

In the discussion of the Graefe method (see p. 14), some results obtained by the procedure of Carius are given.

(*b*) NITRIC ACID DIGESTION.—An obvious simplification of the Carius method is to digest the oil with fuming or concentrated nitric acid at atmospheric pressure, in a Kjeldahl flask or other suitable apparatus. There is no danger of explosions, and when the oil is low in sulphur, a large enough amount to yield a convenient weight of barium sulphate can be taken for analysis.

[18] Liebig's Ann., 116, p. 1, 1860; 136, p. 129, 1865; Ber. d. chem. Ges., 3, p. 697, 1870.

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But the oxidation of the oil proceeds very slowly, so that it may require 2 or 3 days to completely decompose as many grams. In order to hasten the reaction, potassium chlorate has been used.[19] According to the last authors cited, the results on California oils agree well with those obtained by the Carius method.

In this laboratory the use of chlorate was abandoned some years ago because it was found that permanganate hastened the oxidation of the oil. This is due to the formation of manganese peroxide, which spreads through the acid and comes into intimate contact with the oil.

The addition of a few drops of bromine, before the nitric acid is poured into the flask, seems to accelerate the reaction. At any rate it lessens the danger of losing hydrogen sulphide, if the oil contains any.

About a year ago it was found that, although duplicate determinations of the sulphur content of two Mexican oils agreed closely, when the nitric acid-permanganate method was used, yet the results were far too low. By the method described farther on, one of the oils was found to contain 1.83 per cent of sulphur.[20] The other, a topped Tampico crude, contained 4.05 per cent. Analyses by the method just described gave the following results for the two oils: First sample, 0.81, 0.80, 0.81, 0.87, 0.81, and 0.90 per cent; second sample, 2.17, 2.27, and 2.21 per cent.

It was noticed that even when all of the sample appeared to be decomposed, there was always some turbidity and usually oily drops separated out when the contents of the flask were washed into a beaker. This was due to the formation of nitrocompounds which were easily soluble in nitric acid but much less so in water. Although but slightly soluble in water, it was not possible to extract the nitroproduct with any reasonable amount of ether, and it was volatile with steam. The oily product dissolved in sodium-carbonate solution, with which it formed a dark brown salt; it was reprecipitated by acidifying the solution.

The nitrocompounds contained a considerable percentage of sulphur.[21] Failure to decompose the oil completely during the

[19] Kast and Lagai, Dingl. pol. J., 284, p. 71, 1892; Gilpin and Schneeberger, Amer. Chem. J., 50, pp. 65–66, 1913.

[20] This was a topped crude mixed with mineral seal. It was distributed for cooperative analysis by F. R. Baxter, chairman of the subcommittee on sulphur, committee D–2, American Society for Testing Materials. With his permission data taken from his report have been used in the preparation of this paper.

[21] According to Kast and Lagai (see footnote 19) the "resin" formed by the action of nitric acid and chlorate contains no sulphur.

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digestion with acid and permanganate, together with its volatility and solubility, explains why low results for sulphur were obtained. The close agreement between the results actually found is in part accidental, in part due to working with nearly the same amounts of oil and reagents each time, and to stopping the digestion as soon as all oily drops disappeared.

In one determination with nitric acid and permanganate, the digestion was carried on in an all-glass apparatus so constructed that permanganate could be added from time to time. The escaping gases were passed through a solution of permanganate. This soon became decolorized by the oxides of nitrogen, but no sulphate was found in it. In this determination the oil was boiled for 25 hours with the acid, and permanganate was added from time to time. The percentage of sulphur found in the acid was 2.21, as against 2.17 and 2.27 in Kjeldahl flasks.

Goetzl,[22] in a similar experiment with fuming nitric acid, passed the escaping gases through caustic potash solution and found no sulphur.

He also heated fuel oil with fuming nitric acid, until it was all oxidized, then evaporated off the acid, extracted the solid mass with concentrated soda solution, acidified, and filtered off the "bituminous" matter which separated. The amount of sulphuric acid found in the filtrate was too low, but he seems to have missed the point that part of the sulphur is held in the nitrocompounds of which the residue is largely composed. As shown above, and confirmed by Goetzl's statement, this forms a salt with soda and is precipitated by hydrochloric acid.

(*c*) Rothe Method.[23]—The oil is digested in a flask with fuming nitric acid to which a little magnesium oxide has been added. After the acid has boiled for a time it is evaporated off, then more is added and the digestion is continued. This treatment is repeated a few times and finally any oil which remains is burned off with the aid of the magnesium nitrate. The residue in the flask is taken up with hydrochloric acid and water, the solution is filtered, and barium sulphate is precipitated.

The method has not been tested in this laboratory. It has been used elsewhere for the analysis of vulcanized rubber, although it is stated that duplicate results on compounds containing about 7.5 per cent of sulphur may be expected to differ by 0.2 or 0.3 per cent, and they may differ by 0.5 per cent.[24]

[22] Z. angew. Chem., 18, pp. 1528 to 1531; 1905.
[23] Described by Holde in Examination of Hydrocarbon Oils, English translation, pp. 40–41.
[24] Delft Rubber Inst. Communications, Part V, p. 144; 1917.

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On the other hand Holde says that the method gave satisfactory results at the Materialprüfungsamt with oils containing no naphtha or illuminating oil.[25] None of these light fractions had been analyzed at the time he wrote.

(*d*) ESCHKA METHOD.—Originally devised for the determination of sulphur in coal,[26] the method has been applied also to the analysis of asphalt and oils. The sample is mixed with sodium carbonate and magnesium oxide and ignited in a crucible. The residue is treated with bromine water and nitric acid to oxidize all sulphur compounds to sulphuric acid, which is then precipitated as barium sulphate.

It has not seemed worth while to try the method, because of the certainty of losing sulphur as hydrogen sulphide or as volatile organic compounds. This seems to be confirmed by cooperative analyses of three samples of oil.[27] One was a heavy red oil known to contain sulphonic acids, the second a topped Mexican crude mixed with mineral seal, and the third a "300" oil which had been vulcanized by heating with sulphur. The last had a strong odor of hydrogen sulphide.

In one laboratory where Eschka's method was used, the results for the Mexican oil were 1.33 and 1.74 per cent. Five determinations at this Bureau gave from 1.80 to 1.85 per cent.

Two laboratories which used the Eschka procedure for the red oil found from 0.28 to 0.41 per cent. This Bureau found 0.51 to 0.59 in 8 determinations.

The figures on the vulcanized oil, as found in three laboratories, varied from 0.91 to 1.19 per cent; at this Bureau from 1.24 to 1.37 per cent in 8 analyses.

The method used by the Bureau is described further on.

(*e*) SODIUM PEROXIDE METHOD.—In recent years sodium peroxide has been used to a great extent for the determination of the halogens and sulphur. In 1895 Edinger employed it for the estimation of sulphur in volatile organic compounds.[28] The method has not been used for petroleum oils in this laboratory.

2. METHODS INVOLVING COMBUSTION IN OXYGEN OR AIR.

(*a*) SAUER METHOD.[29]—The sample of oil is slowly heated in a combustion tube while oxygen and carbon dioxide are supplied from opposite ends at such a rate that the oil vapors burn at a

[25] The Examination of Hydrocarbon Oils, English translation, p. 40.
[26] Oesterr. Ztschr. Berg- u. Hüttenwesen, **22**, p. 111; Z. anal. Chem., **13**, p. 344, 1874.
[27] See footnote 20, and Goetzl. Z. angew. Chem., **18**, p. 1258; 1905.
[28] Z. anal. Chem., **34**, p. 362; 1895.
[29] Z. anal. Chem., **12**, pp. 32–36; 1873.

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constriction at the middle of the tube. The oxides of sulphur are absorbed in bromine water and barium sulphate is finally precipitated. Even with constant attention it is difficult to keep the flame where it should be and to prevent explosions in the tube, or else the escape of unburned oil. If the oil is low in sulphur, so large a sample must be taken that an analysis requires a long time.

DENNSTEDT METHOD.[30]—The chief feature of the Dennstedt procedure for the combustion of organic substances is the use of a catalyzer in the tube ahead of the boat containing the sample. Platinized quartz or a disk of the sheet metal is commonly used. When determinations of sulphur in oils were first made at this Bureau, the Dennstedt method, with platinized quartz, was used. Because only the sulphur was to be determined, the products of combustion were passed through a Meyer bulb tube containing sodium-hypobromite solution. After repeated trials the method was abandoned because, in spite of constant attention, it was almost impossible to prevent explosions or the escape of unburned gases. Another reason was the long time spent in making a combustion. About two working days was required to burn a 3 g sample. This included the time necessary to heat the catalyzer to redness, to cool the furnace at the end of the first day, and for all the other incidental operations.[31]

(*c*) GRAEFE METHOD.[32]—A small quantity of oil on absorbent cotton is burned in a large (7 l.) bottle of oxygen, in which is some sodium-hydroxide solution to absorb the oxides of sulphur. After the combustion the bottle is rinsed out, the solution is warmed with bromine and acidified, before adding barium chloride. Owing to the great heat developed, only about 0.3 g of oil can be burned, so that the method is not applicable to material low in sulphur.

Two fuel oils were analyzed by the Graefe and Carius methods in the laboratory of the Texas Co., and by a new procedure at this Bureau. Four determinations were made on each oil by each of the first two methods, and but two each by the new one.

The values by the new method were 0.54 and 0.53 per cent for one oil, and 0.76 and 0.77 per cent for the other. By the Carius method the figures varied from 0.62 to 0.65 and from 0.82 to 0.88,

[30] Ber. d. chem. Ges., **30**, pp. 1590–1597; 1902.

[31] In determining nitrogen in petroleum by a combustion method Mabery (J. Amer. Chem. Soc., **41**, p. 1697; 1919) found that "1 g of oil requires two or three hours in safe operation."

[32] Z. angew. Chem., **17**, p. 616; 1904. See also Hempel. Ibid., **5**, p. 393; 1892.

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respectively; by the Graefe procedure, 0.56 to 0.66 and 0.84 to 0.89 per cent of sulphur.

In commenting on these analyses, the chemist of the Texas Co. said that he regarded the results of the three methods as in satisfactory agreement for oils which contained over 0.50 per cent of sulphur.

(*d*) LAMP METHOD.—The sulphur content of kerosene can be determined by burning it in a lamp and drawing the products of combustion through suitable absorption apparatus containing hypobromite solution.[33] The oil must burn with an ample supply of air and without a smoky flame. The air must be free from sulphur compounds. In some unpublished work by the author it was found that reducing gases are present in the products of combustion, and in greater amount the lower the flame. The lamp method for sulphur requires a rather low flame to prevent excessive heating of the absorption apparatus, so that the danger of losing organic compounds containing this element must be borne in mind.

Lubricating oils can be burned in a lamp if they are first mixed with light solvents.[34] According to Conradson, the sulphonic acids remain behind on the wick and can be determined separately. In this connection it should be mentioned that Heusler and Dennstedt claim that the charring of the wick is largely due to the presence of neutral alkyl esters of sulphuric acid. When they break down this acid is formed.[35]

(*e*) BOMB CALORIMETER METHOD.—As early as 1899 the bomb calorimeter was used for the determination of sulphur in oils.[36] Since then it has come to be the procedure most generally employed and the one in which most confidence is placed. The oil is burned in oxygen under high pressure, in a bomb calorimeter which contains a few cubic centimeters of water or of soda solution, or some sodium peroxide.

Even this apparently ideal procedure has certain disadvantages, apart from the cost of the apparatus. Not much more than 1 g of oil can be burned at a time and if the percentage of sulphur is low, very little barium sulphate will be obtained. If a lead gasket is used, it will be appreciably attacked by the oxides of sulphur, which will cause low results. Owing to the high tem-

[33] Heusler, Z. angew. Chem., 1895, p. 285; Engler, Chem.-Ztg., 20, p. 197, 1896.
[34] Albrecht, Diss. Karlsruhe, p. 37, 1907; Conradson, J. Ind. and Eng. Chem., 4, p. 842, 1912; and 8th. Int. Cong. Appl. Chem., 27, p. 19.
[35] Z. anorg. Chem., 17, pp. 264-5; 1904.
[36] Filiti, Bull. soc. chim., Ser. 3, 21, p. 338, 1899; and Z. anal. Chem., 39, p. 727, 1900.

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perature of the combustion, some of the sulphur is burned only to the dioxide. If the bomb is opened too soon this can be tasted in the escaping gases. Under some conditions sulphur trioxide is absorbed slowly by water. A current of air carrying it can be bubbled through water without losing all of its cloudiness.

In the cooperative work referred to above (see p. 13) the three oils were analyzed by the bomb method in several laboratories. The extreme percentages of sulphur were:

	Per cent.
Mexican oil	1.52 to 1.90
Red oil	.39 to .53
Vulcanized oil	.89 to 1.22

One laboratory alone reported five determinations on the red oil which ranged from 0.42 to 0.53 per cent, while eight results for the vulcanized oil varied from 0.89 to 1.19 per cent.

The results obtained at this Bureau are given in the discussion of the Eschka method (see p. 13).

3. MISCELLANEOUS METHODS

In this section are described some attempts to find methods for total sulphur or for the differentiation of the sulphur compounds.

(*a*) HEATING WITH SODIUM.—Although heating with sodium to a comparatively high temperature seems to be a reliable way to test for the presence of sulphur, it could not be depended upon for quantitative data. The sodium was treated with water after the oil was removed with benzene, then bromine water and hydrochloric acid were added, the solution was filtered and finally barium sulphate was precipitated. The results varied greatly.

Metallic calcium under the same conditions appeared to remove no sulphur.

(*b*) HEATING WITH COPPER AND COPPER OXIDE.—Many experiments were made in which the oil was heated with copper powder or with the partially oxidized metal. The powder was prepared by sifting zinc dust into a solution of copper nitrate. The copper was thoroughly washed, finally with alcohol, and dried at a moderate temperature. For some of the tests this powder was heated until it became covered with oxide. The procedure was the same as for sodium, except that the residue, after washing with benzene, was dissolved in dilute hydrochloric acid containing bromine. Considerable sulphur was removed from the oil by this treatment, but the results rarely checked one another.

(*c*) LEAD PEROXIDE.—This reagent was found to remove very little sulphur when heated with oils. In this case the residue was

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extracted with sodium-carbonate solution, as recommended by Dennstedt, or with sodium hypobromite.

(*d*) SODIUM HYPOBROMITE AND HYDROXIDE.—These reagents were used in attempts to remove at least a definite part of the sulphur from the Mexican oil which contained 1.83 per cent of sulphur. A solution of hypobromite, made by adding 1.5 cc of bromine to 50 cc of a cold, 5 per cent caustic soda solution, was shaken with 2.2 to 2.3 g of oil at frequent intervals for three and one-half hours. The mixture was then warmed on the steam bath for one and one-half hours and allowed to stand over night. The aqueous layer was separated and acidified. The addition of barium chloride produced merely traces of precipitate, although the smallest sample taken contained enough sulphur to yield over 290 mg of barium sulphate.

A 5 per cent solution of caustic soda, when heated on the steam bath with the oil and frequently shaken for several hours, extracted only insignificant amounts of sulphur. In this case bromine was added before the hydrochloric acid and barium chloride.

(*e*) DIGESTION WITH PERMANGANATE.—When 0.5 g samples of the Mexican oil which contained 4.05 per cent of sulphur were digested on the steam bath with a strong solution of potassium permanganate for several hours some sulphur was oxidized. In two determinations the amounts of barium sulphate were equivalent to 0.30 and 0.37 per cent, respectively, of sulphur. There is no way, at present, of deciding whether this represented a general attack upon all of the sulphur compounds in the oil, or whether only a particular group of such derivatives was oxidized.

(*f*) DIGESTION WITH HYDROCHLORIC ACID.—This work was done chiefly in order to detect esters of sulphuric acid—the "sulphonic acids," as they are often called. Ten-gram samples of the heavy red oil (see p. 13), which contained about 0.55 per cent of sulphur, were digested on the steam bath with hydrochloric acid of three dilutions, from 1:2 to 2:1. Other portions were boiled with 1:5 acid and still others heated with 1:4 acid in sealed tubes. These were hung inside of a steam bath for several hours. In nine determinations the amounts of sulphuric acid found varied from 0.063 to 0.105 per cent, which corresponds to 0.020 to 0.034 per cent of sulphur.

In the analysis of turkey red oil and other "sulphonated" oils, the usual procedure is to boil with hydrochloric acid. In our tests digestion on the steam bath gave results a few thousandths

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of a per cent higher than those obtained by boiling or by heating in sealed tubes. It is not necessary to boil if the flasks are frequently shaken.

Besides the red oil, the two Mexican oils were also tested. The negligible amounts of sulphuric acid obtained from 10 g samples were equivalent to only 0.001 and 0.004 per cent of sulphur, respectively.

(*g*) HYDROGENATION.—An attempt was made to remove sulphur by heating oil with hydrogen in the presence of a catalyzer. In the first test an engine oil which contained 0.85 per cent of sulphur was used. The apparatus was so arranged that a bulb holding the oil and the catalyzer was heated in an air bath made of an iron crucible with a perforated cover of asbestos board. A slow current of hydrogen bubbled through the oil, passed through a glass tube which was long enough to cool it, and escaped through a Volhard flask containing sodium-hypobromite solution.

The first catalyzer used was made by igniting nickel oxide in hydrogen. The reduced metal was pyrophoric, so that it changed back to oxide before it was mixed with the oil.

At first 3.225 g of oil, with 0.01 g of catalyzer, was heated for two and one-half hours at 320 to 330° C. The hypobromite solution yielded only 0.0023 g of barium sulphate out of a possible 0.1996 g. The same lot of oil was then heated to a much higher temperature, with the thermometer removed, for three hours, after which 0.0043 g more sulphate was obtained. The total of 0.0066 g corresponded to 3.3 per cent of the sulphur in the oil.

Because the topped Mexican crude which contained 1.83 per cent of sulphur gave off streams of hydrogen sulphide when heated, it was tested in the way just described. In five hours' heating, 1.9908 g yielded only 0.0017 instead of a possible 0.2653 g of barium sulphate. When a 7 mm circular disk of platinum foil was used as a catalyzer, 1.7984 g of oil gave 0.0046 g of sulphate.

In the experiments at the higher temperatures the thermometer was removed from the bath when 360° was approached and the flame was increased until the oil vapors condensed 5 or 6 cm above the cover. Only traces of oily distillate were carried over into the hypobromite.[37]

4. NEW METHOD FOR SULPHUR

(*a*) PRELIMINARY WORK.—In a sense all of the methods tested intermittently as opportunity offered since 1908, may be looked

[37] After this work was finished it was learned that Goetzl (Z. angew. Chem., 18, p. 1529; 1905) attempted to determine the sulphur content of fuel oil by distilling off hydrogen sulphide. The amount of this gas which was set free was not proportional to the quantity of oil distilled off.

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upon as leading up to the one finally hit upon. Early in 1910 a series of comparative tests was made of variations of the Henriques method for the determination of total sulphur in rubber. According to the original procedure the rubber is digested with nitric acid, the excess of which is evaporated off. The residue is then oxidized by heating with a mixture of soda and saltpeter. The melt is dissolved in hot water, the solution filtered, acidified and treated with barium chloride. It was found that under certain conditions some sulphur, apparently that in the free state, escaped oxidation by the nitric acid and was lost when the latter was evaporated off. There was no such loss of sulphur when the rubber was treated at the start with nitric acid saturated with bromine.[38]

This success in retaining all of the sulphur in rubber goods led to the use of bromine for the preliminary attack of oils by the nitric acid-permanganate method. At that time, and indeed up to 1919, it was not thought worth while to try the procedure of Henriques for oils, because of the belief that much of the sulphur would be lost by volatilization in organic compounds. This is true if the oil is not first attacked by nitric acid, but it is now known that the nitrocompounds which are formed yield salts when treated with sodium carbonate, and the sulphur is retained.

An oil which six determinations by the nitric acid-permanganate method showed to contain from 0.80 to 0.90 per cent of sulphur, was incorporated with a mixture of soda and saltpeter and carefully ignited. The result was 1.37 per cent. When fused in the same way after treatment with nitric acid and bromine, five determinations gave values from 1.80 to 1.85 per cent. These were the first obtained by the method to be described. A proportionately close agreement is what has come to be expected of beginners in rubber analysis, because there are no complications in the technic.

(*b*) DESCRIPTION OF NEW METHOD.—A preliminary paper describing the method has already been published.[39] The directions here given have been changed but little.

Weigh from 0.5 to 2.0 g of oil in a 100 cc porcelain crucible, add 5 cc of concentrated nitric acid which has been saturated with bromine, and at once cover with a watch glass. When rather volatile oils, or those which smell of hydrogen sulphide are ana-

[38] Waters and Tuttle, B. S. Bulletin, 8, pp. 445–453, 1911; B. S. Scientific Papers No. 174; J. Ind. and Eng. Chem., 3, Oct., 1911. See also Tuttle and Isaacs, B. S. Tech. Papers, No. 45, 1915.

[39] Waters, J. Ind. and Eng. Chem., 12, pp. 482–485 and 612, 1920.

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lyzed, the crucible should be covered during the weighing, and each sample should be treated with the acid at once, and not be left until the series of weighings is finished. Place on the steam bath, but not in direct contact with the steam, and digest for 30 minutes or more, with an occasional gentle swirling of the contents of the crucible. Unless the mixture threatens to foam up to the cover, the crucible may now be heated by the direct steam for two or three hours. If there is much foaming, which happens but seldom, the digestion at moderate heat must be continued longer.

Remove the crucible from the bath, and when its contents have cooled somewhat, add cautiously, in small portions, 10 or 12 g of anhydrous sodium carbonate. This is conveniently measured in a 15 cc crucible, rather than weighed. The first portions of soda often remain on top of the oily layer, so that the crucible must be tilted or swirled slightly to start the reaction. When about half of the soda has been added, rinse the watch glass with 1 or 2 cc of warm water and leave it off. Pour in the rest of the soda. With a glass rod, about 3 mm thick and 12 cm long, thoroughly mix the contents of the crucible and spread the pasty mass around on the sides and half to two-thirds of the way to the top. The rod is left in the crucible, which is then placed on the steam bath, or in an air bath at 100° C.

When its contents are dry enough, the crucible is placed in an inclined position on a wire triangle and the ignition is started over a low flame. When more than 1 g of oil has been taken, there is a tendency for it to burn too briskly, but this is easily controlled by judicious use of the stirring rod which scrapes the burning portion away from the rest. When part of the mass has burned white, more is worked into it, and so on until all of the organic matter is destroyed. It is necessary to hold the crucible with tongs during this operation. Toward the end of the ignition the flame may be increased somewhat, but it is rarely necessary to heat the crucible to redness. With care a good crucible should last for many determinations. Of two which were used for 17 determinations each, and were still in almost perfect condition, one crucible lost just 100 mg, and the other 1.1 mg less.

If a suitable muffle is available, the ignitions can be made at a temperature not above 550°. The glass rod should be removed and the crucible covered with a watch glass. The principal difficulty is that the oil may start to burn at some point and the whole mass begin to glow in a few seconds. In this case the cover

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may be blown aside and loss by spattering occur. Under proper conditions, several ignitions can be made at one time.

According to the amount of oil taken, the mixture of sodium salts resulting from the combustion varies from granular to pasty when hot. After cooling, place the crucible upright in a 400 cc beaker and fill it with distilled water. After a short time lay it on its side, cover the beaker and digest on the steam bath. When the salts have dissolved, remove and rinse the crucible. Filter the solution to remove specks of carbon, bits of the glass rod, etc. Acidify with concentrated hydrochloric acid, using Congo red paper as indicator, and add 2 cc of acid in excess. The neutralization proceeds more safely if nearly the required amount of acid is put in the covered beaker into which the solution is filtered. It is safe to use 10 cc of acid for this, but the exact amount can not be stated, because the total amount of carbonate and nitrite to be neutralized depends on how much oil was taken. The acidified filtrate and wash water should have a volume of about 175 cc. Heat on the steam bath, add 10 cc of barium-chloride solution (containing 100 g of crystallized salt in 1 l), and digest on the bath until the supernatant liquid is clear. If possible delay the filtration until the next day.

A blank should be run with every new lot of reagents. An ignition is made just as if oil were present, because the flame may contain enough sulphur to be corrected for. Even when the correction amounts to several milligrams almost no precipitate may be seen before the solution has stood for several hours.[40]

It is needless to describe how the barium sulphate is filtered off, washed, and ignited.

(*c*) CRITICISM OF NEW METHOD.—It is an advantage that several determinations can be started at one time. The digestions and filtrations for these can be carried on simultaneously. An ignition requires 15 to 25 minutes. (Rubber is so completely broken down that less time is spent in this part of the procedure.) Oil is but slightly decomposed by the treatment with acid, and most of it is driven off in clouds of vapor during the first part of the ignition, and it appears as if much of the sulphur were escaping also. The concordance of duplicate results and their sub-

[40] It is not necessary to have the solution boiling when the barium chloride is added, or to digest the precipitate with the boiling solution. Hillebrand (Analysis of Silicate and Carbonate Rocks, Bull. 700, U. S. Geol. Surv., p. 232) says "at boiling heat or on the steam bath." It is much more convenient to use the steam bath, and there is no danger of loss by bumping; also, as pointed out by Hillebrand, there can be no contamination by sulphur from the flame.

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stantial agreement with analyses by the bomb method are evidence that there can be little or no loss.

In discussing the bomb method a few comparative figures were given. The following were obtained in the analysis of five fuel oils in which the sulphur had been determined by the Bureau of Mines laboratory at Pittsburgh.[41] Each oil was analyzed twice by the new method. The figures in parentheses are the values obtained by the bomb method at the Bureau of Mines. The percentages of sulphur were: 3.67, 3.70 (3.17); 4.01, 4.03 (3.98); 1.13, 1.12 (1.01); 0.36, 0.36 (0.29); 0.18, 0.17 (0.13).

Whatever the faults of the new method, it gives results at least as high as those by the bomb. It has been criticized as too bothersome, yet it does not require the same meticulous attention to details as when the bomb is used. For 10 years it has been used at this Bureau for the analysis of rubber, and even in the hands of beginners has given good results. For oils it may well be employed in laboratories which do not have sufficient work of this kind to justify the expense of a bomb calorimeter.

(*d*) SOURCES OF ERROR.—The most obvious way in which incorrect results can be obtained—failure to convert all of the sulphur into sodium sulphate—appears not to be a fault of this method. It has, however, one serious fault in common with all of the other procedures in which salts or oxides of metals are used to effect or facilitate the oxidation of the sulphur compounds in the oil. This is the occlusion of the salts of these metals by the barium-sulphate precipitate. Even when barium chloride is added to water which contains only sulphuric acid, the precipitate is not pure, but is contaminated by more or less chloride, and the results are too high. If soluble sulphates, particularly those of sodium, potassium, or ammonium, are present, there is a considerable amount of occlusion and during the ignition of the precipitate a complication of reactions, with the net result that the values found are too low. On the other hand, the presence of nitrates causes errors in the opposite direction. Finally, the amount of dissolved barium sulphate which is lost in the filtrate and washings depends on their volume and on the quantity of free acid, other than sulphuric.[42]

[41] They were obtained through the interested cooperation of E. W. Dean and N. A. C. Smith, the latter in the Washington laboratory of that Bureau.

[42] For an interesting discussion of the determination of sulphur as barium sulphate, see the following papers; Allen and Johnston, J. Am. Chem. Soc., **32**, pp. 588–617, 1910; Johnston and Adams, Ibid., **33**, pp. 829–845. 1911.

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(*e*) CORRECTION FOR OCCLUSION.—By adapting to the present purposes a suggestion made by Johnston and Adams in the paper just cited, data were obtained which show the necessary corrections of weights of barium sulphate up to 0.200 g. In order to have definite amounts of sulphur, a solution of pure sodium sulphate which was dried at 200° C before weighing out, was prepared. By taking various aliquot portions of this solution, weights of barium sulphate from 3 to 200 mg were obtained. Without going into all the details, the procedure was as follows: To check the purity of the sodium salt an aliquot portion was diluted to 175 cc, acidified with 2 cc of concentrated hydrochloric acid, heated on the steam bath and precipitated with 10 cc of the usual barium-chloride sulution. The results were in satisfactory agreement with those calculated.

In an actual determination of the sulphur in oil, the aqueous solution of the melt contains sodium carbonate, nitrate, nitrite, and sulphate. When this is acidified, the carbonate and nitrite are decomposed, so that finally only chloride, nitrate, and sulphate are present. The amount of chloride depends indirectly upon the weight of oil taken. This is true also of the residual nitrate. In order to have approximately the same conditions as when amounts of oil varying from 0.5 to 2.0 g were burned, portions of sodium carbonate equal to those used in an analysis were dissolved and treated with 2, 3, or 4 cc of concentrated nitric acid. Then to the solution was added hydrochloric acid until there was 2 cc in excess beyond the neutral point. The desired aliquot portion of sulphate solution was added, the volume made up to 175 cc, and barium sulphate precipitated as usual. Next day the precipitates were filtered off and finally weighed. Blanks were run under the same conditions, only with the sodium sulphate left out. The small amounts found in the blanks were used as corrections for the larger weights of barium sulphate.

No matter how much sodium sulphate was taken, the weights of barium sulphate found, after being corrected for the impurities in the reagents, were greater than those calculated from the weights of sodium sulphate. The differences between the weights found and those calculated were the corresponding corrections for occlusion and solubility. On plotting these corrections against the weights of barium sulphate found, it was seen that a straight line running from the origin to a point with the coordinates 10 and 200 mg represented the facts with sufficient accuracy. In

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other words, the net error amounted to 5 per cent of the weight of the precipitate. The variations from this straight line were so slight as to be of no significance in the analysis of oils. The most surprising fact was that changing the volume of nitric acid caused so little difference in the amount of occlusion.

With a 1 g sample, 200 mg is equivalent to 2.75 per cent of sulphur and the corrected weight, 190 mg, to 2.61 per cent. Under other conditions of volume, acidity, etc., the corrections, and the errors if these are neglected, might be very different.

An easy way to find the weights of sulphur corresponding to the amounts of barium sulphate found is by means of a graph. Figure 1 is such a graph drawn from the data obtained in the determinations of the occluded salts. Each centimeter on the axis of abscissas represents 10 mg of barium sulphate. On the axis of ordinates each centimeter represents 1 mg of sulphur. After correction, 200 mg of barium sulphate becomes 190 mg, which corresponds to 26.1 mg of sulphur. From the point represented by the coordinates 200 and 26.1 draw a straight line to the origin. For any weight of sulphate up to 200 mg the weight of sulphur can be read off directly, and its percentage calculated. If the same weight of oil, say 1 g, is taken for analysis each time, the percentages, instead of the weights of sulphur can be laid off on the axis of ordinates; thus instead of 26.1 mg there would be 2.61 per cent. It is not always desirable to use so little or so much as 1 g of oil, and it may be advisable to make one calculation rather than to delay long enough to make an exact weighing if the oil is losing hydrogen sulphide. For these reasons it seems preferable to lay off the ordinates as first stated.

An example from an actual analysis is given: 0.5053 g of oil yielded 0.0722 g of barium sulphate. The sodium carbonate was so poor that the correction for it was 0.0085 g. This subtracted from the first weight gave 0.0637 g. On the graph 63.7 mg of barium sulphate is seen to be equivalent to 8.3 mg of sulphur. This divided by 0.5053 gives 1.64 per cent. The percentage calculated without the graph is 1.66, an amply satisfactory agreement.

After the manuscript of the present paper was completed, and a few days before a preliminary paper was published in the Journal of Industrial and Engineering Chemistry (May, 1920, pp. 482 to 485), it was learned that an almost identical method was published 15 years ago. This earlier paper seems to have been entirely overlooked or ignored by oil chemists. Goetzl

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(Z. angew. Chem., 18, pp. 1528 to 1531; 1905) used the following procedure for the determination of sulphur in "liquid fuel": In a roomy platinum crucible treat 2 or 3 g of oil with 4 cc of fuming nitric acid. The crucible, covered with a watch glass, is allowed

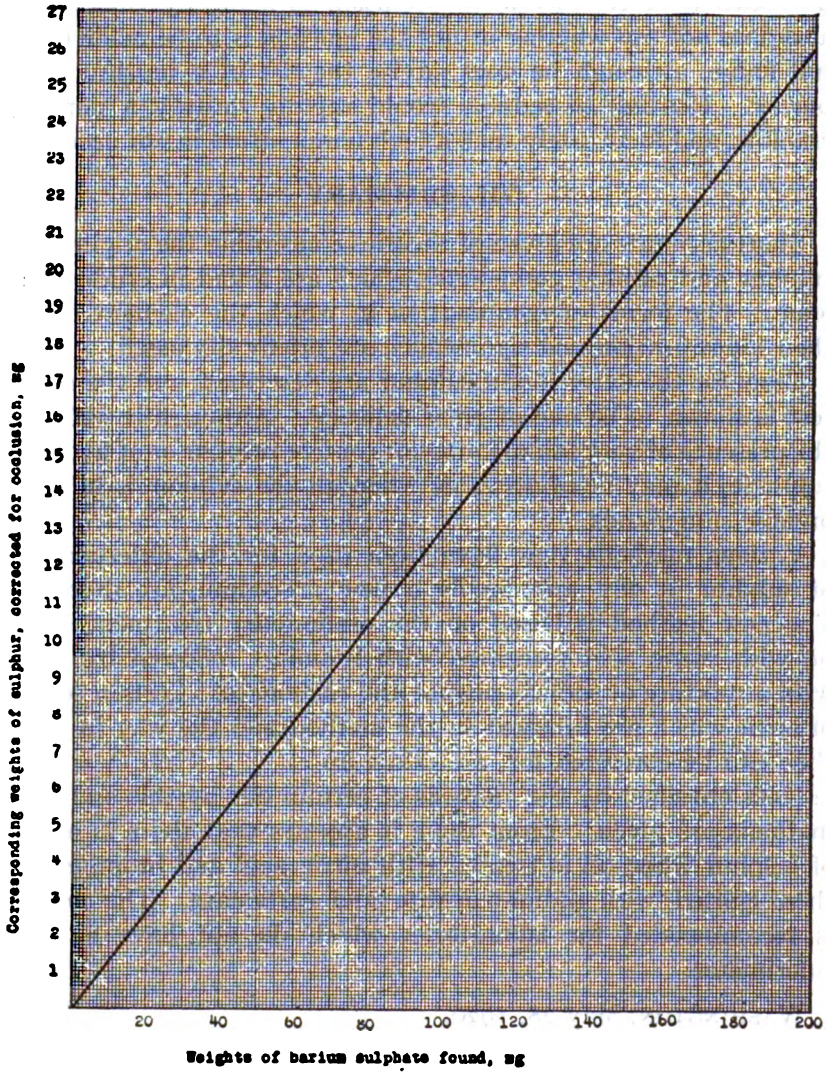

FIG. 1.

to stand over night and then warmed gently on a water bath. The acid is evaporated off and the oily nitrated mass is mixed with sodium carbonate and potassium nitrate. More of the fusion mixture is spread over the top and the whole is ignited. The subsequent operations need not be described.

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Kerosene and naphtha contain so little sulphur that 10 g samples were taken for analysis. The method was used for coal, lignite, peat, organic substances, etc.

As in the majority of papers on the determination of sulphur, no mention of errors due to occlusion is made by Goetzl. He also does not say whether or not the ignition is to be made over a full flame, although he does advise the use of a mushroom-top burner. Heating to redness would rapidly destroy a porcelain crucible, and fortunately such a high temperature in not necessary.

VII. SUMMARY

Short accounts are given of theories concerning the origin of the sulphur and sulphur compounds which are found in crude petroleum. The forms of combination in which the element occurs, their identification and significance are briefly discussed.

Tests for the detection of sulphur are described, and the copper test is shown to be one of great delicacy. Although it will show the presence of very minute amounts of free sulphur or of hydrogen sulphide, it may be of no value when all the sulphur is in stable organic compounds.

Various methods that have been used for the determination of sulphur in oils, and finally a new procedure, are described. The new method is based on the preliminary treatment of the oil with nitric acid saturated with bromine, followed by fusion with a mixture of sodium nitrate and carbonate. Certain sources of error which are inherent in the method and the way in which they can be corrected for, are taken up.

Data obtained by the analysis of certain oils by this and other methods are given. From these it appears that there is no loss of sulphur when oil is treated with nitric acid and bromine, and then ignited with sodium nitrate and carbonate. The method is recommended for laboratories which do not have a bomb calorimeter.

WASHINGTON, May 28, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 178

STEEL RAILS FROM SINK-HEAD AND ORDINARY RAIL INGOTS

BY

GEORGE K. BURGESS, Physicist

Bureau of Standards

DECEMBER 15, 1920

PRICE, 20 CENTS

Sold only by the Superintendent of Documents, Government Printing Office, Washington, D. C.

WASHINGTON
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1921

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By George K. Burgess

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I. GENERAL PLAN OF INVESTIGATION

The results of the experimental rolling into rails of several types of ingots, including several designs of sink-head ingots and others, with the coöperation of the Maryland Steel Co., reported by Hadfield and Burgess,[1] looked sufficiently promising to the rail committee of the Pennsylvania Railroad to warrant the purchase of 100 tons of ingots cast by Sir Robert Hadfield in England, in order to make a more effective study of the suitability of this type of sink-head ingot for the manufacture of rails. It was also considered desirable to carry out similar series of tests of ingots of ordinary rail steel rolled at the same time and under as nearly identical conditions as possible in the same mill.[2] President F. W. Wood, of the Maryland Steel Co., Sparrows Point, Md., offered the facilities of that plant for this investigation, as in the case of previous experiments reported by Hadfield and Burgess. The rolling of ingots for the present investigation was done in 1915 at the Sparrows Point plant. The reduction of the results has been greatly delayed, but it is thought the subject matter is still of sufficient interest to warrant publication.

The planning of the details of the investigation, after correspondence with Sir Robert Hadfield, was made by A. W. Gibbs, chief mechanical engineer of Pennsylvania Railroad Co.; F. W. Wood, president of Maryland Steel Co.; and Dr. G. K. Burgess, chief, division of metallurgy, Bureau of Standards; and the tests were carried out by members of the technical staffs of their respective organizations, to whom acknowledgment is here made for their indispensable aid.

It was expected by Sir Robert Hadfield that the sink-head ingots made under this system would yield rails of remarkably high quality as regards "soundness, freedom from all piping and all segregation." From an inspection of the data concerning the manufacture of the ingots made by this process he also felt con-

[1] Hadfield and Burgess, Sound Steel Ingots and Rails, Trans. Am. Inst. Min. Engrs., 51, pp. 862–880, 1915; J. Iron and Steel Inst., 91, pp. 40–124, 1915.

[2] See also the valuable paper comparing American-made sink-head and ordinary ingots for rails by Edward F. Kenney: The Commercial Production of Sound and Homogeneous Steel, Am. Iron and Steel Inst., p. 144, 1915.

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fident in stating: "This shows in a remarkable manner the uniformity of the ingots; also that every one of them is perfectly sound, free from blowholes, with no piping, and segregation below 8 per cent discard."

The results of the present investigation of 37 ingots of the Hadfield type show that he was not far astray in the confident prediction of uniformity, physical soundness, and freedom from chemical segregation. In fact, it is difficult to imagine 37 ingots, from as many heats of steel, more uniform in quality and properties and as free from those undesirable qualities and uncertainties that beset the ordinary ingot from which rails are made.

The ingots with which they were compared, the product of three heats, 5 from each heat, or 15 ingots in all, are not less instructive, in that each group, or heat, represents a distinct practice in open-hearth manufacture. In this investigation, therefore, from the point of view of steel making, we are really comparing the products rolled from 2 types of ingot representing 4 methods of manufacture.

The rolling, inspection, tests, and analyses were carried out with the greatest care, and by a series of actual weighings all the material from the several ingots was accounted for.

II. DETAILED SCHEME OF PROCEDURE

The detailed scheme of observations of rolling and tests of sink-head and comparison ingots follows:

1. INGOTS

METHOD OF NUMBERING INGOTS.—The sink-head ingots were numbered H1, H2, etc., ... H37, and the ingots made in the ordinary manner were numbered M1, M2, etc., ... M15.

WEIGHT OF INGOTS.—The sink-head ingots were weighed cold and the others were weighed hot. The approximate weight of the sink-head ingots was 5300 pounds each and of the comparison ingots 7300 pounds.

SIZE AND SHAPE OF INGOTS.—The English-made ingots were cast with sink head on large end, while the American-made ingots were cast small end up; dimensions of both types of ingot are shown in Table 1, and details concerning the sink heads of Hadfield ingots are to be found in Table 2.

COMPOSITION OF INGOTS.—Each sink-head ingot was from a separate heat, the compositions of which, furnished by the manufacturer, are given in Table 2. The American-cast ingots were from three heats, and their chemical compositions are shown in Table 3.

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TEMPERATURE OF INGOTS.—The temperature of pouring of the comparison ingots is shown in Table 6; the temperature of the sink-head ingots was taken when they were introduced into the soaking pits, the temperatures of which were also noted then and at intervals until the ingots were withdrawn from the soaking pits; the temperature of English (Table 7) and comparison (Table 8) ingots were also taken at the blooming mill. In Table 9 are shown typical ingot-rolling temperatures of both types of ingot.

INSPECTION OF INGOTS.—The sink-head ingots were inspected cold for surface defects, cracks, etc., and at the blooming mill during rolling for the existence and location of cracks, scrap, etc. (Table 4). The other ingots were similarly inspected at the blooming mill (Table 5).

DISPOSITION OF INGOTS.—Of the 37 sink-head ingots cast at Sheffield, 35 were rolled into blooms at the Sparrows Point mill, and the remaining 2 ingots were sectioned longitudinally for metallographic examination. The examination of H32 was made by the Bureau of Standards and that of H37 by Hadfields (Ltd.). In addition to splitting 2 ingots, it was also decided to examine similarly top blooms from one of each of the two types of ingot, H5 and M10. The blooms and ingots examined in this manner were selected at random. All blooms were rolled into rails directly without reheating, and all material was accounted for by actual weighing, as shown in Tables 10 and 11.

TABLE 1.—Dimensions of Ingots of Both Types

Part measured	Sink-head ingot	Comparison ingot
	Inches	Inches
Top	19½ by 19½	18¼ by 18¼
Bottom	17¼ by 17¼	19¼ by 19¼
Height	[a] 50½	63

[a] The height given is exclusive of the height of sink head, which was approximately 13¾ inches.

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TABLE 2.—Results of Chemical Analyses and Head Particulars of Sink-Head Ingots [a]

Ingot number	Heat number	Chemical composition (percentages)					Sink-head particulars			
		C	Si	S	P	Mn	Height to which filled in head	Depth of settling	Capacity of cavity	Percentage of settling to total bulk of ingot
							Inches	Inches	Cm^3	
H1	4774	0.65	.....	0.040	.....	0.94	14.00	8.25	9950	3.19
H2	4738	0.63–.64	.....	.042	.....	.91	13.50	9.00	10300	3.31
H3	4847	.63–.64	.....	.047	.....	.93	13.75	9.25	10450	3.37
H4	4915	.65–.66	.....	.049	.....	.92	14.00	9.75	9675	3.10
H5	4873	.69	.....	.050	.....	.95	14.00	9.38	10250	3.29
H6	4939	.63–.64	0.16	.045	0.028	.92	13.00	7.50	8800	2.84
H7	4627	.64	.....	.052	.....	.97	14.00	8.25	9600	3.08
H8	4680	.65	.14	.049	.029	.92	13.75	8.50	10250	3.29
H9	4709	.58	.....	.049	.....	.93	13.00	7.75	8930	2.88
H10	5085	.63–.64	.16	.042	.031	.86	13.25	9.25	11000	3.52
H11	5018	.64	.16	.039	.031	.92	14.00	8.75	10040	3.21
H12	4641	.67–.69	.....	.045	.....	.92–.94	14.50	8.50	9000	2.89
H13	5059	.63–.64	.....	.049	.....	.92	14.00	8.88	10750	3.44
H14	4835	.69	.....	.046	.....	.91	13.50	7.75	9520	3.05
H15	4978	.67–.68	.....	.051	.....	.96	14.50	9.88	9260	2.95
H16	4992	.66	.....	.045	.....	.94	13.50	9.75	9000	2.88
H17	4825	.66	.....	.045	.....	.92	13.50	6.88	9000	2.89
H18	4829	.66	.....	.041	.....	.98	14.00	8.50	11100	3.55
H19	5073	.63–.64	.....	.045	.....	.90	14.00	9.50	10500	3.37
H20	4696	.66	.....	.038	.....	.96	13.50	6.50	8600	2.77
H21	4801	.66	.....	.036	.....	.95	13.50	8.13	9150	2.90
H22	4723	.63	.....	.044	.....	.95	13.50	8.00	9900	3.17
H23	5046	.62–.63	.....	.042	.....	.93	13.50	7.00	10500	3.37
H24	5030	.60	.....	.056	.....	.94	13.50	10.13	11950	3.82
H25	4668	.58	.....	.042	.....	.93	14.50	8.50	10100	3.24
H26	4813	.64	.14	.048	.032	.93	14.00	8.63	10800	3.45
H27	4615	.64	.20	.048	.....	1.01	14.25	7.00	9000	2.89
H28	4588	.64	.19	.047	.....	.97	13.00	7.25	8930	2.86
H29	4908	.64	.....	.041	.....	.84	14.00	8.75	9750	3.11
H30	4861	.61–.62	.21	.045	.031	.92	14.00	9.63	10900	3.49
H31	4600	.65	.15	.049	.....	.96	13.50	8.50	9825	3.16
H32	5004	.65	.....	.052	.....	.94	14.00	8.50	10175	3.27
H33	4761	.65	.....	.048	.....	.90	13.50	8.50	10800	3.47
H34	5099	.63–.64	.....	.039	.....	.86	11.00	8.88	10040	3.34
H35	4887	.63	.....	.050	.....	.89	14.00	9.00	10500	3.36
H36	4653	.61	.....	.053	.....	.85	13.50	7.75	9900	3.17

[a] Furnished by Sir Robert Hadfield.

TABLE 3.—Results of Chemical Analyses of Maryland Steel

Heat number	Ingot numbers	Elements determined (percentages)						
		C	Si	S	P	Mn	Cr	Ni
2 x 4497	M1 –M5, inc.	.608–0.630	0.148	0.067	0.010	0.710–0.720	0.270	0.760
1 x 3632	M6 –M10, inc.	.642–.654	.185	.092	.014	.650–.660	.290	.780
2 x 4510	M11–M15, inc.	.642–.650	.191	.097	.019	.760–.810	.270	.760

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TABLE 4.—Results of Inspection of Sink-Head Ingots and Blooms

Ingot number	Results of inspection of cold ingots for cracks, scrap, etc.	Time of charging ingots into pits	Time of drawing ingots from pits	Interval in pits	Time consumed in rolling in blooming mill	Results of inspection during rolling in bloomimg mill
					Min. sec.	
24	1 side—bad splash in pouring, 1 rough corner	4:35 p. m. to 4:40 p. m., Aug. 15, 1915.	11:56 a. m. to 12:25 p. m., Aug. 16, 1915.	19 hrs., 32 mins.	2 10	Very small piece of scrap on one corner.
28	1 side—bad splash in pouring bottom half, 1 rough corner				2 22	Very small piece of scrap on sides and corners.
2	1 side rough, 2 rough corners				2 25	1 crack 18 inches from top on side of ingot.
15	1 side very rough near top, 2 rough corners				2 15	2 small cracks on corners 2 feet from top of ingot.
1	Sides good, 2 rough corners				2 10	Spot of scrap 2 feet from top.
21	1 rough side, 1 rough corner				2 20	Spot of scrap 2 feet from top. Small crack on corner 2 feet from top.
9	3 rough sides, 1 rough corner	4:45 p. m. to 4:52 p. m., Aug. 15, 1915.	12:29 p. m. to 12:46 p. m., Aug. 16, 1915.	19 hrs., 49 mins.	2 25	Cracked all around ingot at bottom of sink-head.
7	Sides O. K., 2 rough corners				2 15	Small spot of scrap 8 inches from top on corners.
35	3 sides—rough spots 1 foot from bottom, 1 rough corner				2 35	Small spots of scrap all over ingot.
13	3 rough sides, 2 rough corners				2 20	Small spots of scrap 12 inches from bottom.
22	1 side—bad splash in pouring, 1 rough corner				2 10	Small spots of scrap on corners 2 feet from bottom.
14	2 sides—splashed in pouring, 3 rough corners				2 20	Clean bloom.
19	1 side—bad splash in pouring, 2 rough corners	4:30 p. m. to 4:38 p. m., Aug. 16, 1915.	1:49 p. m. to 2:05 p. m., Aug. 17, 1915.	21 hrs., 22 mins.	1 30	Small spot of scrap on corner 1 foot from top.
26	1 side rough 1 inch from top, 2 rough corners				2 15	Small spots of scrap 2 feet from top.
18	1 side—bad splash in pouring, corners good				2 15	Cracked at bottom of sink-head.
6	2 sides—bad splash in pouring, 2 rough corners				2 20	1 small spot of scrap 4 feet from top and one 1 foot from bottom on sides.
23	2 sides—bad splash in pouring, 1 rough corner				2 25	
31	1 side—bad splash in pouring near bottom, 2 rough corners				2 30	1 spot of scrap 1 foot from top on side. Clean bloom. Clean bloom.
29	1 side—crack in mold 10 inches from top, 2 rough corners. 2 sides—rough spots in center	4:50 p. m. to 5:05 p. m., Aug. 16, 1915.	2:06 p. m. to 2:22 p. m., Aug. 17, 1915.	21 hrs., 15 mins.	2 40	Spots of scrap on corners and sides 18 feet from top.
36	2 sides—rough spots 1 foot from bottom, corners good				.. ..	
4	4 sides—splash in pouring, 2 rough corners				2 20	Small spot of scrap 2 feet from top.
11	2 sides—splash in pouring, 2 rough corners				2 15	Spots of scrap at center across the corners.
12	1 side—splash in pouring, corners good				2 20	Small spots of scrap all over the top third of ingot.
10	Sides good, 3 rough corners				2 25	Small spots of scrap 2 feet from top on corners and sides.
16	2 sides—rough 2 feet from bottom, 1 rough corner	4:30 p. m. to 4:45 p. m., Aug. 17, 1915.	9:51 a. m. to 10:10 a. m., Aug. 18, 1915.	17 hrs., 22 mins.	2 30	Small spots of scrap 3 feet from bottom on corners.
20	2 sides rough, 3 rough corners				2 20	Small spot of scrap 1 foot from top on corner.
33	3 sides—bad splash in pouring 1 foot from top, 1 rough corner				2 15	Clean bloom. Cracked at bottom of sink-head.
17	1 side—splash in pouring, 2 rough corners				2 20	Small spot of scrap at center on sides.
30	3 sides—splash in pouring at center, 2 rough corners				2 20	Large spots of scrap on all sides.
27	1 side—splash in pouring 2 feet from bottom, 1 rough corner				2 15	Small spots of scrap 2 feet from top on sides.
					2 30	Small spots of scrap on all sides. Cracked at bottom of sink-head.
8	1 side—splash in pouring 6 inches from bottom, 1 rough corner.	4:45 p. m. to 5:00 p. m., Aug. 17, 1915.	10:14 a. m. to 10:27 a. m., Aug. 18, 1915.	17 hrs., 28 mins.	2 25	Large spot of scrap at center of ingot on one side.
3	2 rough sides, 2 rough corners				2 40	Small spot of scrap 2 feet from top.
25	3 sides—splash in pouring, 1 rough corner				2 20	Large spot of scrap 2 feet from top on corner.
34	1 side rough at center, 2 rough corners				2 50	Large spots of scrap on all sides.
5	1 side—splash in pouring, corners good				2 25	Small spots of scrap on sides.
32	1 side—splash in pouring, 2 rough corners				.. ..	Not rolled.

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TABLE 5.—Time Consumed in Rolling and Results of Inspection of Comparison Ingots and Blooms

Ingot number	Date rolled	Heat number	Time in molds	Time in soaking pits	Time consumed in rolling in blooming mill	Results of inspection during rolling in blooming mill
			Min.	H. M.	M. S.	
1	Aug. 16, 1915	2x4497	20	2 10	2 25	1 corner and 1 side cracked 1 foot from bottom.
2	do	2x4497	20	2 14	1 50	Small spot of scrap 2 feet from top on corner.
3	do	2x4497	20	2 18	2 35	Clear bloom.
4	do	2x4497	20	2 21	2 ..	Do.
5	do	2x4497	20	2 23	2 ..	Do.
6	Aug. 18, 1915	1x3632	23	1 40	1 50	Two small side cracks 4 feet from bottom.
7	do	1x3632	23	1 40	1 45	Clear bloom.
8	do	1x3632	23	1 41	2 10	Do.
9	do	1x3632	23	1 43	1 55	Do.
10	do	1x3632	23	1 45	2 10	1 small side crack 5 feet from bottom.
11	do	2x4510	30	2 15	2 ..	Clear bloom.
12	do	2x4510	30	2 15	2 ..	Small side crack 3 feet from top.
13	do	2x4510	30	2 17	2 ..	Clear bloom.
14	do	2x4510	30	2 19	2 10	Small side crack 3 feet from top.
15	do	2x4510	30	2 20	2 5	Small side crack 2 feet from bottom.

TABLE 6.—Pouring Temperatures of Comparison Ingots [a]

Observation number	Temperature for heat 1x3632	Temperature for heat 2x4510	Observation number	Temperature for heat 1x3632	Temperature for heat 2x4510
	°C	°C		°C	°C
1	.....	1557	16	1543	1522
2	.....	1545	17	1533	1512
3	[b] 1596	[c] 1545	18	1533	1507
4	[b] 1587	[c] 1538	19	1539	1497
5	[b] 1564	[c] 1540	20	1523	.....
6	[b]	[c] 1528	21	1539	.....
7	[b] 1578	[c] 1531	22	1537	.....
8	1559	1532	23	1537	.....
9	1557	1537	24	1529	.....
10	1557	.....			
11	1549	1527			
12	1557	1531			
13	1547	1521			
14	1551	1533			
15	1539	1527			

[a] Pouring temperatures group M_1–M_5, inclusive, not taken.
[b] Ingots from group M_6–M_{10}, inclusive. (Average pouring temperature, 1581°C.)
[c] Ingots from M_{11}–M_{15}, inclusive. (Average pouring temperature, 1536°C.)

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TABLE 7.—Temperatures of Working of Sink-Head Ingots

Ingot number	Temperature of soaking pits		Ingot temperature		Rail temperature at finishing pass (taken on base of rail)	
	At time of charging ingots	After 14½ hours	When drawn	Blooming mill	First rail bar	Second rail bar
	°C	°C	°C	°C	°C	°C
1	700	1310	1300	1130	1016	1007
2	700	1310	1300	1140	1030	1020
15	700	1310	1300	1137	1026	1017
21	700	1310	1300	1123	1025	1005
24	700	1310	1300	1172	1025	1010
28	700	1310	1300	1157	1019	1008
7	750	1300	1290	1125	1028	998
9	750	1300	1290	1149	1024	1016
13	750	1300	1290	1116	1034	1008
14	750	1300	1290	1143	1043	1019
22	750	1300	1290	1144	1032	1018
35	750	1300	1290	1140	1034	1011
6	650	1200	1295	1149	1048	1025
18	650	1200	1295	1166	1032	1023
19	650	1200	1295	1180	1069	1044
23	650	1200	1295	1154	1047	1026
26	650	1200	1295	1173	1047	1034
31	650	1200	1295	1146	1036	1016
4	650	1240	1280	1156	1040	1016
10	650	1240	1280	1147	1046	1022
11	650	1240	1280	1136	1029	1012
12	650	1240	1280	1142	1047	1029
29	650	1240	1280	1148	1034	1005
36	650	1240	1280	1136	1038	1016
16	600	1305	1515	1182	1018	955
17	600	1305	1515	1164	1036	1016
20	600	1305	1515	1179	1045	1023
27	600	1305	1515	1153	1040	1024
30	600	1305	1515	1157	1032	1020
33	600	1305	1515	1160	1029	1017
3	600	1300	1316	1156	1033	1023
5	600	1300	1316	1149	.....	.....
8	600	1300	1316	1151	1029	1023
25	600	1300	1316	1153	1035	1000
34	600	1300	1316	1140	1014	1004
Average	658	1276	1299	1150	1034	1014
Maximum	750	1310	1315	1182	1069	1044
Minimum	600	1200	1280	1116	1014	955

TABLE 8.—Temperatures of Working of Comparison Ingots

Ingot number	Ingot temperature in blooming mill	Rail temperature at finishing pass (taken on base of rail)	
		First rail bar	Second rail bar
	°C	°C	°C
1	1149	1026	1006
2	1173	1016	993
3	1164	1040	1025
4	1179	1053	1039
5	1177	1064	1027
6	.....	1031	1005
7	1167	1029	1018
8	1176	1030	1001
9	1159	1038	1009
10	1180	1038	.....
11	1162	1039	1003
12	1181	1046	1025
13	1161	1032	1018
14	1149	1025	1016
15	1149	1024	1018
Average	1166	1035	1001
Maximum	1181	1064	1039
Minimum	1149	1016	993

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TABLE 9.—Typical Rolling Temperatures

Temperature of sink-head ingot number							Temperature of comparison ingot number					
7	13	14	19	28	30	34	2	5	7	10	13	15
°C	°C	°C	°C	°C	°C	°C	°C	°C	°C	°C	°C	°C
1136	1157	1152	1212	1181	1195	1165	1184	1173	1179	1206	1179	1168
1147	1114	1147	1212	1171	1179	1165	1171	1184	1179	1198	1171	1157
1144	1103	1136	1176	1152	1165	1157	1168	1176	1176	1177	1168	1136
1114	1109	1147	1176	1147	1147	1141	1184	1176	1160	1179	1157	1149
1123	1109	1141	1160	1147	1147	1131	1171	1179	1163	1173	1157	1147
1125	1128	1141	1165	1147	1139	1123	1165	1184	1157	1168	1147	1136
1112	1093	1139	1165	1147	1144	1120	1163	1179	1157	1163	1147	1144
1103	.....	.....	.....	.....	1141	1125	.....	1147	.....	.....	.....	.....
Average							Average					
1125	1116	1143	1180	1157	1157	1141	1173	1177	1167	1180	1161	1149

	°C		°C
Grand average	1145	Grand average	1168
Maximum	1212	Maximum	1206
Minimum	1093	Minimum	1136

TABLE 10.—Disposition of Material from Sink-Head Ingots

[Weight in pounds. Discard made only to physically sound steel. H_5 not included in this table as the top bloom for this ingot was sent to the Bureau of Standards]

Ingot number	Ingot weight	Bloom crops				Weight of the four test pieces each 6 inches long	Rail crops at hot saw		
		Top		Bottom			Middle		Bottom
		Weight	Per cent of ingot, average	Weight	Per cent of ingot, average		No. 1	No. 2	
ROLLED AUG. 16, 1915									
24	5300	185	3.5	297	5.6	86	77	60	259
28	5300	233	4.4	112	2.1	79	74	32	30
2	5300	155	2.9	113	2.1	79	66	30	21
15	5300	167	3.2	129	2.4	79	83	31	48
1	5300	190	3.4	123	2.3	77	59	38	30
21	5300	442	8.4	116	2.2	85	43	77	75
9	5300	294	5.5	131	2.5	79	46	75	89
7	5250	234	4.5	122	2.3	79	54	54	74
35	5300	280	5.3	161	3.0	77	46	48	50
13	5300	191	3.6	141	2.7	81	58	39	58
22	5250	437	8.3	123	2.3	81	47	45	54
14	5300	231	4.4	138	2.6	78	53	50	67
ROLLED AUG. 17, 1915									
19	5300	160	3.0	120	2.3	77	49	50	69
26	5300	160	3.0	130	2.5	75	47	53	70
18	5300	237	4.5	116	2.2	76	50	77	99
6	5300	324	6.1	127	2.4	77	44	56	69
23	5350	134	2.5	131	2.4	78	49	79	100
31	5300	234	4.4	109	2.1	74	54	28	49
29	5350	566	10.6	123	2.3	77	44	63	78
36	5300	342	6.5	142	2.7	76	50	47	71
4	5350	270	5.0	123	2.3	71	50	46	56
11	5300	325	6.1	136	2.7	73	73	39	61
12	5350	204	3.8	124	2.3	74	54	60	97
10	5250	320	6.1	130	2.5	78	46	57	61
ROLLED AUG. 18, 1915									
16	5350	199	3.7	154	2.9	74	37	56	79
20	5300	276	5.2	122	2.3	73	43	66	72
33	5300	228	4.3	148	2.8	75	49	78	93
17	5300	145	2.7	117	2.2	74	53	73	93
30	5300	247	4.7	112	2.1	74	45	79	117
27	5300	245	4.7	97	1.9	78	57	63	71
8	5300	142	2.7	119	2.3	78	53	75	86
3	5300	205	3.8	115	2.2	71	81	55	65
25	5300	160	3.0	151	2.8	77	64	57	72
34	5300	195	3.7	166	3.2	83	58	43	57
Total	179 700	8357	4.3	4519	2.5	2623	1856	1879	2540

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TABLE 10—Disposition of Material from Sink-Head Ingots—Continued

ROLLED AUG. 16, 1915—Continued

Ingot number	Ingot weight	Weight of rails				Total weight rolled, including crops and test pieces	Loss	
		A	B	C	D		Weight	Per cent of ingot, average
24	5300	768	1111	1117	1122	5082	218	4.1
28	5300	1232	1112	1120	1121	5145	155	2.9
2	5300	1315	1119	1120	1123	5141	159	3.0
15	5300	1277	1115	1120	1123	5172	128	2.4
1	5300	1284	1116	1122	1123	5162	138	2.6
21	5300	941	1121	1121	1124	5145	155	2.9
9	5300	1096	1121	1117	1119	5167	133	2.5
7	5250	1203	1116	1119	1122	5177	73	1.4
35	5300	1172	1113	1115	1118	5180	120	2.3
13	5300	1289	1116	1118	1120	5211	89	1.7
22	5250	1032	1116	1117	1122	5174	76	1.4
14	5300	1228	1113	1115	1117	5190	110	2.1

ROLLED AUG. 17, 1915—Continued

19	5300	1175	1108	1112	1115	5035	265	5.0
26	5300	1226	1108	1110	1112	5091	209	3.9
18	5300	1130	1107	1111	1115	5118	182	3.4
6	5300	1035	1107	1111	1111	5061	239	4.5
23	5350	1183	1102	1105	1109	5070	280	5.2
31	5300	1191	1100	1110	1117	5066	234	4.4
29	5350	867	1107	1111	1111	5147	203	3.8
36	5300	1082	1106	1106	1113	5135	165	3.1
4	5350	1228	1106	1107	1111	5168	182	3.4
11	5300	1161	1107	1116	1114	5205	95	1.8
12	5350	1194	1101	1107	1114	5129	221	4.1
10	5250	1077	1110	1114	1115	5108	142	2.7

ROLLED AUG. 18, 1915—Continued

16	5350	1191	1111	1111	1115	5127	223	4.2
20	5300	1175	1107	1111	1113	5158	142	2.7
33	5300	1153	1107	1112	1112	5155	145	2.7
17	5300	1269	1107	1113	1115	5159	141	2.7
30	5300	1151	1111	1112	1113	5161	139	2.6
27	5300	1229	1119	1120	1121	5200	100	1.9
8	5300	1274	1117	1109	1112	5165	135	2.5
3	5300	1204	1103	1106	1109	5114	186	3.5
25	5300	1275	1104	1109	1112	5181	119	2.2
34	5300	1157	1105	1115	1113	5092	208	3.9
Total	179 700	39 464	37 749	37 859	37 946	174 791	5519	3.0

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TABLE 10.—Disposition of Material from Sink-Head Ingots—Continued

ROLLED AUG. 16, 1915—Continued

Ingot number	Ingot weight	Discard in rail on account of—			Material discarded to physically sound steel		Material available for rails			Ingot rolled with no draft on
		Sink head	Pipe	Total	Weight	Per cent of original ingot, average	Weight	Per cent of original ingot, average	Net per cent of ingot, average	Pass number
24	5300	161	.....	161	346	6.5	4043	76.3	87.8	1
28	5300	163	.....	163	396	7.5	4501	84.9	89.2	1
2	5300	230	.....	230	385	7.3	4526	85.4	88.9	1
15	5300	310	.....	310	477	9.0	4404	83.1	88.1	2
1	5300	292	.....	292	482	9.1	4430	83.5	87.8	2
21	5300	162	.....	162	604	11.1	4230	79.8	84.8	2
9	5300	.....	.....	.....	294	5.5	4532	85.5	91.8	1
7	5250	235	.....	235	469	8.9	4404	83.9	88.2	1
35	5300	160	.....	160	440	8.3	4435	83.7	88.6	1
13	5300	80	.....	80	271	5.1	4644	87.6	92.7	2
22	5250	.....	.....	.....	437	8.3	4468	85.1	88.8	2
14	5300	284	.....	284	515	9.7	4367	82.4	97.5	2

ROLLED AUG. 17, 1915—Continued

19	5300	326	.....	326	486	9.2	4261	80.4	85.0	1
26	5300	233	.....	233	393	7.4	4398	83.0	87.9	1
18	5300	.....	324	324	561	10.5	4215	79.5	85.0	1
6	5300	158	.....	158	482	9.1	4283	80.8	85.5	2
23	5350	291	.....	291	425	7.9	4286	80.1	86.7	2
31	5300	237	.....	237	471	8.9	4355	82.2	86.0	2
29	5350	64	.....	64	630	11.8	4209	78.7	84.3	1
36	5300	73	.....	73	415	7.8	4410	83.2	88.4	1
4	5350	87	162	249	519	9.7	4374	81.8	87.0	1
11	5300	254	.....	254	579	10.9	4317	81.5	86.4	2
12	5350	367	.....	367	571	10.7	4223	78.9	85.1	2
10	5250	148	.....	148	468	8.9	4346	82.8	86.8	2

ROLLED AUG. 18, 1915—Continued

16	5350	305	.....	305	504	9.4	4297	80.4	86.4	1
20	5300	313	.....	313	589	11.1	4266	80.5	85.4	1
33	5300	281	.....	281	509	9.6	4278	80.7	86.7	1
17	5300	342	.....	342	487	9.1	4336	81.8	87.4	2
30	5300	270	.....	270	517	9.8	4291	80.9	86.8	2
27	5300	233	.....	233	478	9.0	4434	83.7	88.5	2
8	5300	346	.....	346	488	9.1	4344	82.0	87.5	1
3	5300	378	.....	378	583	11.0	4215	79.5	84.6	1
25	5300	340	.....	340	500	9.4	4337	81.8	87.5	1
34	5300	389	.....	389	584	11.0	4184	78.9	84.1	2
Total	179 700	7512	486	7998	16 355	9.1	147 643	81.9	87.9	.....

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TABLE 11.—Disposition of Material from Comparison Ingots

[Weights in pounds. Discard made only to physically sound steel]

HEAT NO. 2x4497

Ingot number	Ingot weight	Bloom crops				Weight of the five test pieces, each 6 inches long
		Top		Bottom		
		Weight	Per cent of ingot, average	Weight	Per cent of ingot, average	
1	7360	183	2.4	276	3.7	90
2	7030	243	3.4	209	2.9	98
3	7040	185	2.6	143	2.0	100
4	7130	189	2.6	226	3.1	95
5	7190	223	3.1	175	2.4	101
	35 750					

HEAT NO. 1x3632

6	7340	189	2.5	291	3.9	93
7	7200	205	2.8	215	2.9	90
8	7260	188	2.5	215	2.9	90
9	7240	225	3.1	252	3.4	93
	29 040					

HEAT NO. 2x4510

11	7510	664	8.8	236	3.1	96
12	7340	288	3.9	254	3.4	89
13	7190	490	6.8	175	2.4	88
14	7320	379	5.1	210	2.8	95
15	7480	236	3.1	235	3.1	96
	36 840					
Total	101 630	3887	3.8	3112	3.0	1314

HEAT NO. 1x3632

10		3315		230		97

HEAT NO. 2x4497—Continued

Ingot number	Ingot weight	Rail crops at hot saw			Weight of rails					
		Middle		Bottom	X	A	B	C	D	E
		No. 1	No. 2							
1	7360	48	58	71	930	1117	1117	1117	1123	1126
2	7030	84	96	94	517	1114	1115	1117	1120	1122
3	7040	95	92	74	675	1110	1109	1115	1117	1119
4	7130	147	81	37	660	1108	1108	1115	1116	1120
5	7190	89	100	59	774	1108	1109	1113	1115	1119
	35 750									

HEAT NO. 1x3632—Continued

6	7340	44	63	75	878	1118	1118	1121	1121	1123
7	7200	34	56	28	880	1116	1114	1117	1122	1122
8	7260	60	59	73	888	1111	1112	1122	1120	1120
9	7240	23	59	94	783	1118	1118	1122	1123	1125
	29 040									

HEAT NO. 2x4510—Continued

11	7510	36	87	145	591	1102	1106	1111	1110	1116
12	7340	48	76	115	826	1105	1103	1106	1108	1111
13	7190	42	29	123	625	1101	1101	1104	1104	1106
14	7320	37	80	127	775	1099	1099	1104	1101	1103
15	7480	69	70	123	1011	1103	1106	1107	1109	1108
	36 840									
Total	101 630	856	1006	1238	10 813	15 530	15 535	15 591	15 609	15 640

HEAT NO. 1x3632—Continued

10			68	140				1115	1116	1117

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TABLE 11.—Disposition of Material from Comparison Ingots—Continued

HEAT NO. 2x4497—Continued

Ingot number	Ingot weight	Total weight rolled, including crops and test pieces	Discard in rail on account of pipe	Material discarded to physically sound steel		Material available for rails			Date rolled
				Weight	Per cent of original ingot, average	Weight	Per cent of original ingot, average	Per cent of net ingot, average	
1	7360	7256	505	688	9.3	6115	83.1	88.5	Aug. 16, 1915
2	7030	6929	.....	243	3.4	6203	88.2	94.8	Do.
3	7040	6934	.....	185	2.6	6345	90.1	95.8	Do.
4	7130	7002	336	525	7.4	5986	83.9	90.2	Do.
5	7190	7085	.....	223	3.1	6439	89.5	95.0	Do.
	35 750	.....	.....	1864	5.2	31 088	86.9	92.8	

HEAT NO. 1x3632—Continued

6	7340	7234	505	694	8.2	6067	82.6	88.3	Aug. 18, 1915
7	7200	7099	505	710	9.8	6056	84.1	88.2	Do.
8	7260	7158	163	351	4.8	6400	88.1	93.5	Do.
9	7240	7135	336	561	7.7	6146	84.8	90.4	Do.
	29 040	.....	.....	2316	8.0	24 669	85.0	90.2	

HEAT NO. 2x4510—Continued

11	7510	7400	1515	2179	29.0	4717	62.8	67.4	Aug. 19, 1915
12	7340	7229	336	624	8.5	6112	83.2	89.2	Do.
13	7190	7088	2356	2846	39.5	3873	53.8	56.7	Do.
14	7320	7209	1682	2061	28.2	4694	64.1	68.3	Do.
15	7480	7373	1815	2051	27.4	4825	64.5	69.1	Do.
	36 840	.....	.....	9761	26.5	24 221	65.8	70.2	
Total	101 630	100 131	10 054	13 941	13.7	79 978	78.7	83.8	

HEAT NO. 1x3632—Continued

10	.....	7180	.....	.....	.....	.....	.....	.....	Aug. 18, 1915

BLOOMING MILL.—All ingots were rolled in 13 passes in a two-high mill to blooms 8 by 8⅛ inches. The English made ingots entered the blooming mill with the sink head toward the rolls. One half received the first, or "squaring up," pass in this direction, no draft being given on second pass; the other half were given no draft on the first pass, the "squaring up" being done on pass No. 2 (Table 10). The comparison ingots were rolled according to ordinary practice, entering the rolls the small end first.

2. BLOOMS

Each ingot was cut into 2 blooms (8 by 8⅛ inches) as shown in Figs. 1 and 2, and rolled directly into rails, that is, without reheating of blooms. The second bloom from sink-head ingots made 2 rails (C and D), and that from the comparison ingots made

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3 rails (C, D, and E). The first bloom from the sink-head type of ingot made a B and, generally, a short A rail; the first bloom from the comparison type of ingot made a B, A, and (short) X rail. The blooms were sheared top and bottom as little as possible for the safe passage of the bar through the rolls. Each piece cut was marked, weighed, and preserved.

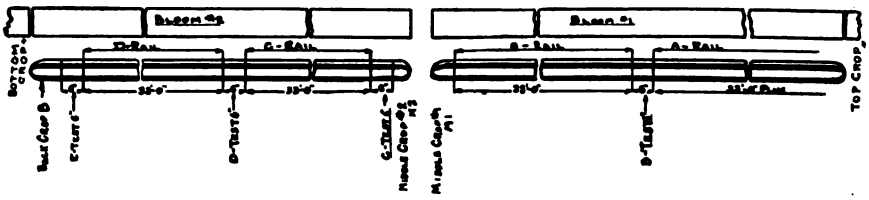

FIG. 1.—*Method of cutting blooms and rails from sink-head ingots*

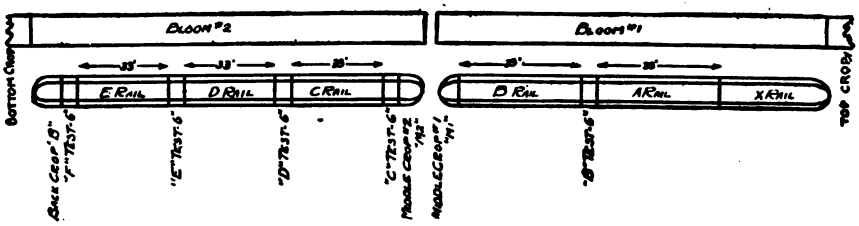

FIG. 2.—*Method of cutting blooms and rails from comparison ingots*

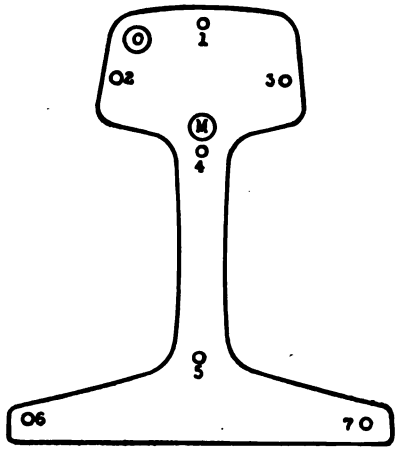

FIG. 3.—*Section of 100-pound P. S. rail, showing location of various* tests

Drillings for chemical analysis were taken at "O" and "M." Hardness determinations were made at the other positions indicated

3. RAILS

In the rail mill there are 6 passes in the roughing rolls, 4 in the intermediate rolls, and 1 finishing pass, making a total of 24 passes

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from ingot to finished rail. The rails, which were Pennsylvania 100-pound section (Fig. 3), were cut at the hot saw as shown in Figs. 1 and 2.

The temperature of all rails was taken at the finishing pass. The rails were inspected for surface defects, piping, and discard to physically sound steel. The classification into "firsts" and "seconds" from surface conditions was made by the Pennsylvania Railroad Co. inspectors (Table 12); the discard to "physically sound steel" was made by cutting back in 5-foot lengths.

TABLE 12.—Classification of Rails Rolled from Sink-Head and Comparison Ingots [a]

Rails from sink-head ingots				Rails from comparison ingots			
Ingot number	B rail	C rail	D rail	B rail	C rail	D rail	E rail
1	1	2	1	1	1	1	2
2	1	1	1	1	1	1	1
3	1	2	1	1	1	1	1
4	2	2	2	1	1	1	1
5	(a)	2	2	1	1	1	2
6	2	2	2	1	1	1	1
7	2	2	2	1	1	1	1
8	2	2	2	1	1	1	1
9	1	1	2	1	1	1	1
10	2	1	2	(b)	1	1	1
11	2	2	2	1	1	1	2
12	2	1	2	1	1	1	1
13	2	2	2	(b)	2	1	1
14	1	1	1	(b)	1	1	1
15	2	2	2	1	1	1	1
16	2	2	2				
17	1	1	2				Per ct.
18	1	1	1	No. 1 rails		53	93
19	1	1	1	No. 2 rails		4	7
20	2	2	2	Total		57	100
21	1	2	1				
22	1	1	1				
23	1	1	2				
24	1	1	1				
25	2	2	2				
26	1	1	1				
27	2	2	2				
28	2	2	2				
29	1	2	1				
30	2	2	2				
31	1	2	1				
32	2	2	2				
33	1	2	2				
34	2	2	2				
35	1	1	1				

		Per ct.
No. 1 rails	43	41
No. 2 rails	61	59
Total	104	100

[a] Inspection made by Pennsylvania Railroad Co. inspectors.
[b] Sent to Bureau of Standards.

The HA, the MX, and MA rails were held for tests to be described later. All pieces of rail were stamped to identify the ends.

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4. TESTS

MECHANICAL TESTS.—The first sound 5-foot length obtained was taken as the drop-test piece. The test was made in the usual way by dropping a 2000-pound tup from a height of 18 feet upon the head of the rail which was supported at 2 points 3 feet apart. This test gives record of permanent set, elongation of the base, and the number of blows required for the destruction of the rail.

The 6-inch test pieces (B, C, D, etc., Figs. 1 and 2), the drop-test pieces, and the specimens from the MA position were examined for ultimate tensile strength, yield point, elongation (2-inch gage length), reduction of area, and hardness by Brinell and scleroscope. All specimens for the tension tests were cut longitudinally.

METALLOGRAPHIC TESTS.—There were taken of each test-piece section, a sulphur print, a light etching, and two photomicrographs at 100 diameters. Photomicrograph A was taken at upper part of web, B corresponds approximately with O position (Figs. 8 to 12, inclusive). These tests were also made on sections of the 5-foot pieces of the MX, MA, and HA rails to accompany the determinations of discard for chemical segregation.

CHEMICAL TESTS.—Determinations were made of the amounts of carbon, sulphur, phosphorus, manganese, and silicon at the "O" and "M" positions (Fig. 3) for test pieces (B, C, D, etc.) of all the rails; samples from the 5-foot pieces of MX, MA, and HA were also analyzed. Some specifications contain a clause which provides for the rejection of rails when the M position shows a carbon content more than 12 per cent greater (positive segregation) or more than 12 per cent less (negative segregation) than the O position. In the present investigation, all material showing more than 12 per cent positive or negative carbon segregation was considered as unavailable for rails. The railroad using this specification has found from experience that 12 per cent is the maximum safe limit of segregation.

III. DESCRIPTION OF OPERATIONS AND DISCUSSION OF RESULTS

1. INGOT PRACTICE

Since the sink-head ingots were cast in England and rolled in this country, they were necessarily subjected to the unusual process, at least in rail manufacture as ordinarily practiced, of reheating from the cold condition. The plan was considered of

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allowing at least some of the comparison ingots to go "stone cold" and reheat them before rolling. However, in view of the fact that the ordinary type of ingot is not designed to meet this condition, which practically never obtains in practice and which, therefore, would be a possible misleading condition to impose on the comparison ingots and of no practical value, the comparison ingots were all rolled directly after being cast. These ingots were stripped from 20 to 30 minutes after casting and put into the soaking pits while still hot, where they remained from 1 hour and 40 minutes to 2 hours and 23 minutes.

The sink-head ingots after preliminary heating were charged into relatively cool pits (for temperature details see Section III, 2, Temperature Observations). The heat was gradually turned on after ½ hour and the ingots remained in the pits from 17⅓ hours to 21⅓ hours. The 35 sink-head ingots which were rolled were divided into three series and rolled on successive days, August 16, 17, 18, 1915. Three heats of comparison ingots were rolled, 1 on August 16 and 2 on August 18. There was no delay or other unusual condition in the mill during the rolling. The ingots of a group were rolled in succession, without intermission, and after the mill had been warmed up by rolling at least one complete heat of the regular product of the mill.

2. TEMPERATURE OBSERVATIONS

Temperature observations were taken of the soaking pits before, and at intervals, after immersion of ingots of both types; of all ingots in the blooming mill; and of rails from both series of ingots at the finishing pass. In order to heat them gradually, the Hadfield ingots lay on the surface of the soaking pits approximately 12 hours before charging. They were too hot to bear the hand comfortably when charged, at which time their estimated temperature was about 50° C (122° F). The pits which received these sink-head ingots were allowed to cool to the temperature of 600° C (1112° F)—750° C (1382° F) (Table 7). The immersion of the ingots further cooled the pits by some 200° C (392° F) as measured immediately after charging. The gas was turned on the pits ½ hour after charging. The comparison ingots were charged into the soaking pits hot, according to the usual practice.

Table 7 shows the values of the pit temperatures (1) before charging, (2) about 14½ hours after charging, and (3) just before drawing the sink-head ingots. These temperatures were taken with a platinum-rhodium thermocouple connected to a pivot

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type of indicator. The couple was inclosed in nickel sheath except the tip, which was exposed. In some cases the temperatures were obtained by the use of an optical pyrometer of the Morse type.

By means of the optical pyrometer temperatures of the ingots were taken during the rolling. Average values obtained from 7 or 8 observations with optical pyrometer at alternate passes, after the first roughing pass, were taken of the ingots in the blooming mill. The temperature of the steel at the surface was lowered about 17° C (63° F) to 40° C (104° F) during the rolling into blooms. Typical illustrations of rolling temperatures are given in Table 9 for several ingots of both types. Table 7 contains finishing temperatures of Hadfield rail bars taken at the finishing pass by sighting on the center of base of rail bar with an optical pyrometer. Each observation is the average of two readings.

In Table 8 are similar data for the comparison ingots and rails, except that the pit temperatures were not taken. It will be noted that good uniformity of rolling and finishing temperatures was obtained.

A series of measurements was taken to compare the temperatures of this investigation taken at the finishing pass with those taken [3] at the hot saw by sighting on the head of the rail. The reading noted in Tables 7 and 8 of finishing temperatures should be reduced by about 11° C (52° F) to correspond to previous observations at hot saw.

The observed ingot and rail temperatures were taken on the outside oxidized surfaces, which are colder than the interior metal by 75° C (167° F) to 100° (212° F). This would account for the apparent discrepancy between the soaking-pit and ingot temperatures, and would make the average true finishing temperature somewhat higher than 1100° C (2012° F).

3. GENERAL COMPARISON OF THE TWO TYPES OF INGOT

(*a*) Size of Ingots.—The sink-head ingots were of 5200 pounds weight and the comparison ingots of 7300 pounds. In computing the percentage of ingot material available for rails the necessary constant valued deductions—i. e., the bottom-bloom crops and the middle and bottom rail crops—work to the relative disadvantage of the ingot of lesser weight. In was on this account that in Tables 10 and 11 the percentage of material available for

[3] B. S. Tech. Papers, No. 38, Observations on finishing temperatures and properties of rails.

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rails is based upon both the original weight of ingot less the weight of the bottom-bloom crops and the initial weight minus the middle and bottom rail crops.

(*b*) LOSS IN HEATING.—The sink-head ingots were weighed while cold before reheating, and the comparison ingots were weighed while hot. The abnormal loss in weight sustained by the former should be deducted from the nominal ingot weight in computing the net ingot. It so happens that the average bottom-bloom crop for the ingot made in an ordinary manner is 0.5 per cent greater than for the sink-head ingot; so that if the loss on heating for the comparison ingots is taken as 0.5 per cent there will be an exact balance. The English-cast ingots remained in the soaking pits some 10 times as long as the comparison ingots. If it is nevertheless preferred to compare actual percentages of material available for rails, that from the sink-head ingots should be increased by 2.5 per cent because there was a loss of 3 per cent, of which some 2.5 per cent can be charged to reheating. When this correction is made it is seen that the average sink-head ingot gives about 84.8 per cent material available for rails instead of 81.9 per cent. Ingot H24 should not be used in the discussion, as it was accidentally given an excessive bloom crop. The expression "material available for rails," as used here and in Tables 10 and 11, refers to only physically sound steel. Subsequent chemical analyses showed a much greater discard was necessary to eliminate segregation above 12 per cent (Tables 17 and 18).

(*c*) COMPARISON BY HEATS.—It will be noted that the three heats of comparison ingots have each very distinct characteristics; for example, as expressed in material available for rails, which are the results of varying open-hearth and casting practice. In group M1–M5 the percentage of material available for rails was 86.9 per cent; in group M6–M9, 85 per cent; and in group M11–M15, 65.8 per cent (Table 11). It was for this reason that, in addition to the general average of properties of all Maryland rails, averages were computed for each of the three Maryland heats. The three rollings of the Hadfield ingots show great uniformity, however expressed, and thus indicate that the manufacture of the ingots in all cases apparently was carried out in accordance with a uniform practice.

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TABLE 13.—Mechanical and Chemical Characteristics of Rails from Sink-Head Ingots

Rail No.	Average, ADFM,[a] or DFAR [b]	Tension tests				Hardness		Elements determined (per cent)									
		Yield point	Ultimate strength	Elongation, per cent in 2 inches	Reduction of area	Brinell	Scleroscope	Carbon			Manganese		Phosphorus		Silicon		Sulphur	
								O	M	Segregation[c]	O	M	O	M	O	M	O	M
		Lbs. in.	Lbs. in.		Per cent													
HA	Average	62 074	125 986	14.5	23.48	259	33	0.652	0.733	+12.73	0.88	0.85	0.035	0.033	0.137	0.139	0.047	0.050
	ADFM[a]	1 585	2 342	0.7	1.25	6	1	.018	.044	7.84	.04	.03	.003	.002	.008	.007	.003	.005
	DFAR[b]	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
HB	Average	63 212	126 381	13.3	23.89	264	31	.649	.664	2.32	.89	.89	.032	.032	.139	.139	.047	.048
	ADFM	1 206	2 027	0.9	1.50	8	1	.016	.018	1.35	.03	.04	.002	.002	.011	.012	.004	.005
	DFAR	+1 138	+395	−1.2	+0.41	+5	−2	−.003	−.069	−10.41	+.01	+.04	−.003	−.001	+.002	0	0	−.002
HC	Average	62 941	126 054	13.6	24.30	260	31	.653	.644	− 1.27	.89	.88	.032	.031	.139	.138	.047	.046
	ADFM	1 109	2 130	0.7	1.18	9	1	.017	.017	1.68	.03	.04	.002	.002	.011	.012	.003	.003
	DFAR	867	+68	−0.9	+0.82	+1	−2	+.001	−.089	−14.00	+.01	+.03	−.003	−.002	+.002	−.001	0	−.004
HD	Average	63 029	126 158	13.6	24.47	260	30	.650	.621	4.48	.89	.88	.032	.031	.139	.137	.047	.044
	ADFM	1 310	2 381	0.7	1.14	7	1	.017	.022	1.81	.03	.03	.002	.002	.011	.012	.004	.004
	DFAR	+955	+172	−0.9	+0.99	+1	−3	−.002	−.112	−17.21	+.01	+.01	−.003	−.002	+.002	−.002	0	−.006
HE	Average	62 264	124 303	14.2	25.70	256	30	.637	.639	+ 0.40	.89	.88	.032	.037	.137	.137	.046	.046
	ADFM	1 418	2 900	0.6	1.63	10	1	.019	.023	1.42	.03	.04	.002	.003	.011	.011	.004	.004
	DFAR	+190	−1 683	−0.3	+2.22	−3	−3	.015	.094	12.33	+.01	+.03	−.003	−.001	0	−.002	−.001	−.004
GENERAL AVERAGES																		
H	General averages	62 704	125 771	13.8	24.38	260	31	0.648	0.660	1.94	0.89	0.88	0.032	0.033	0.138	0.138	0.047	0.047
	ADFM	1 326	2 356	0.8	1.34	8	1	.017	.025	2.75	.03	.04	.002	.002	.010	.011	.004	.005

[a] Average deviation from mean.
[b] Deviation from A rail.
[c] Shows percentage carbon variation of M from O position.

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TABLE 14.—Mechanical and Chemical Characteristics of Rails from Comparison Ingots

Rail No.	Average, ADFM[a] or DFAR[b]	Tension tests				Hardness		Carbon		
		Yield point	Ultimate strength	Elongation, per cent in 2 inches	Reduction of area	Brinell	Sclero-scope	O	M	Seg.[c]
		Lbs./in.²	Lbs./in.²		Per cent					
M1-5A	Average	60 721	120 960	13.5	23.16	246	32	0.601	0.736	+22.32
	ADFM[a]	821	2 928	1.0	3.23	5	1	.023	.056	6.65
M6-9A	Average	60 115	123 600	12.8	21.42	252	33	.652	.730	+11.88
	ADFM	885	1 075	1.0	1.61	8	0	.009	.045	6.04
M11-15A	Average	64 963	129 695	13.3	23.43	267	34	.682	.738	+ 8.52
	ADFM	1 322	1 226	0.2	1.50	7	1	.150	.058	9.60
M1-5B	Average	64 145	127 910	11.6	20.80	260	33	.641	.709	+10.60
	ADFM	1 048	2 048	0.5	1.61	3	1	.011	.041	4.78
M6-9B	Average	63 140	128 188	11.6	20.18	259	33	.675	.715	+ 6.18
	ADFM	238	938	0.6	1.20	6	1	.008	.095	1.51
M11-15B	Average	64 484	130 140	12.5	23.92	263	33	.661	.671	− 1.53
	ADFM	345	888	0.4	0.70	1	1	.070	.190	1.49
M1-5C	Average	61 752	124 290	12.5	24.18	244	32	.634	.613	− 3.40
	ADFM	411	548	0.4	0.57	7	1	.007	.030	4.09
M6-10C	Average	61 982	127 520	12.0	20.81	255	34	.673	.673	0.00
	ADFM	593	556	0.6	0.56	2	1	.011	.021	2.12
M11-15C	Average	62 415	126 850	13.1	26.77	354	32	.673	.646	− 4.02
	ADFM	704	1 720	0.5	0.71	3	1	.070	.050	0.89
M1-5D	Average	61 254	123 295	13.1	26.70	245	32	.635	.596	− 6.12
	ADFM	1 207	1 016	0.7	1.40	5	1	.011	.016	1.04
M6-10D	Average	61 158	125 700	12.5	23.75	246	31	.671	.635	− 5.43
	ADFM	686	940	0.4	0.50	2	0	.003	.009	1.56
M11-15D	Average	62 290	126 750	13.6	26.95	252	31	.674	.607	− 9.88
	ADFM	772	1 060	0.3	0.26	4	1	.080	.070	1.58
M1-5E	Average	59 233	120 465	14.7	30.40	240	31	.623	.534	−14.35
	ADFM	1 150	722	0.6	1.30	3	0	.003	.010	1.32
M6-10E	Average	59 000	123 055	13.9	26.72	246	33	.650	.617	− 4.99
	ADFM	446	874	0.4	1.44	2	1	.008	.013	2.15
M11-15E	Average	60 944	124 050	14.3	29.26	242	31	.643	.570	−11.40
	ADFM	1 372	1 020	0.4	0.55	6	1	.110	.110	1.25

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M1-5F	Average	67 592	117 190	15.5	32.34	234	32	.585	.595	+ 1.73
	ADFM	500	1 048	0.6	0.59	8	1	.004	.033	6.07
M6-10F	Average	59 395	121 735	15.2	30.03	244	32	.637	.643	− 0.53
	ADFM	382	1 248	0.5	0.73	5	2	.007	.008	2.05
M11-15F	Average	59 450	120 065	15.8	32.68	249	32	.613	.611	− 0.39
	ADFM	1 028	1 502	0.2	0.82	2	0	.040	.080	1.42

SUMMARY BY INGOT POSITION

M-A rails	Average	62 063	124 834	13.2	22.76	255	33	0.644	0.735	+14.41
	ADFM	2 091	3 472	.6	2.14	11	1	.064	.045	7.53
	DFAR	0	0	0	0	0	0	0	0	0
M-B rails	Average	63 979	128 786	11.9	21.73	260	33	.665	.697	+ 5.00
	ADFM	840	1 488	.5	1.19	4	1	.032	.109	2.67
	DFAR	+ 1 916	+3 952	− 1.3	− 1.03	+ 5	0	+ .021	− .038	− 9.41
M-C rails	Average	62 050	126 220	13.5	23.92	251	33	.660	.644	− 2.42
	ADFM	631	1 606	.6	.61	6	1	.029	.034	2.35
	DFAR	− 13	+ 1 388	+ .3	+ 1.16	− 4	0	+ .016	− .091	−11.99
M-D rails	Average	61 567	125 248	13.1	25.80	248	31	.660	.612	− 7.14
	ADFM	999	1 495	.6	.72	4	1	.031	.032	1.39
	DFAR	− 496	+ 414	− .1	+ 3.04	− 7	− 2	+ .016	− .123	− 7.27
M-E rails	Average	59 726	122 523	14.3	28.79	243	32	.638	.573	−10.40
	ADFM	1 068	1 536	.5	1.09	5	1	.043	.044	1.57
	DFAR	− 2 337	− 2 311	+ 1.1	+ 6.03	− 12	− 1	− .006	− .162	− 4.01
M-F rails	Average	58 826	119 663	15.5	31.68	242	32	.611	.616	+ .27
	ADFM	1 052	2 049	.5	.71	8	1	.017	.040	3.18
	DFAR	− 3 237	− 5 179	+ 2.3	+ 8.92	−13	− 1	− .033	− .119	−14.14

SUMMARY BY HEATS

M1-5	Average	60 783	122 352	13.5	26.26	245	32	0.620	0.631	+ 1.80
	ADFM	1 873	3 068	1.2	3.53	9	1	.020	.064	10.84
M6-10	Average	60 757	124 792	13.1	24.03	250	33	.636	.665	+ .98
	ADFM	1 416	2 151	1.2	3.13	6	1	.024	.038	5.64
M11-15	Average	62 431	126 258	13.8	27.17	255	32	.661	.641	− 3.22
	ADFM	1 604	4 681	.7	2.62	3	1	.038	.042	5.14

GENERAL AVERAGE

	General average	61 368	124 545	13.6	25.78	250	32	0.645	0.646	+ 0.16
	ADFM	1 113	1 941	.6	1.01	6	1	.036	.051	3.12

[a] Average deviation from mean. [b] Deviation from A rail. [c] Shows percentage carbon variation of "M" from "O" position.

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TABLE 14.—Mechanical and Chemical Characteristics of Rails from Comparison Ingots—Continued

Rail No.	Average, ADFM[a] or DFAR[b]	Manganese		Phosphorus		Silicon		Sulfur		Nickel		Chromium	
		O	M	O	M	O	M	O	M	O	M	O	M
M1-5A	Average	0.64	0.68	0.010	0.013	0.127	0.130	0.058	0.086	0.75	0.75	0.28	0.29
	ADFM[a]	.01	.02	.001	.001	.005	.006	.004	.015	.01	.01	.01	.01
M6-9A	Average	.66	.67	.015	.018	.142	.145	.096	.124	.72	.72	.29	.30
	ADFM	.01	.01	.000	.002	.007	.008	.007	.018	.00	.01	.00	.00
M11-15A	Average	.77	.79	.028	.032	.178	.179	.118	.117	.73	.73	.26	.27
	ADFM	.01	.02	.001	.004	.015	.018	.004	.029	.00	.01	.00	.00
M1-5B	Average	.68	.69	.012	.013	.129	.130	.074	.097	.77	.77	.28	.28
	ADFM	.01	.01	.000	.000	.007	.006	.004	.011	.01	.00	.00	.02
M6-9B	Average	.68	.68	.016	.018	.168	.147	.103	.115	.75	.75	.28	.28
	ADFM	.01	.01	.001	.001	.020	.002	.001	.006	.01	.01	.01	.01
M11-15B	Average	.78	.78	.030	.029	.174	.174	.122	.120	.73	.74	.27	.27
	ADFM	.01	.01	.001	.001	.007	.007	.001	.006	.00	.01	.01	.00
M1-5C	Average	.67	.67	.013	.012	.128	.128	.076	.073	.77	.78	.27	.28
	ADFM	.01	.00	.001	.001	.007	.006	.001	.005	.00	.01	.01	.01
M6-10C	Average	.66	.66	.017	.017	.145	.145	.101	.103	.74	.74	.26	.29
	ADFM	.01	.01	.000	.001	.004	.003	.001	.002	.01	.01	.01	.01
M-11-15C	Average	.78	.78	.029	.027	.173	.172	.118	.114	.74	.74	.27	.27
	ADFM	.01	.02	.000	.001	.008	.008	.003	.002	.00	.00	.01	.00
M1-5D	Average	.67	.66	.012	.016	.127	.145	.007	.094	.78	.74	.29	.29
	ADFM	.01	.01	.001	.001	.008	.001	.003	.004	.00	.01	.01	.01
M6-10D	Average	.67	.67	.016	.012	.144	.127	.100	.069	.74	.78	.29	.28
	ADFM	.00	.00	.001	.001	.002	.008	.002	.003	.00	.01	.01	.01
M11-15D	Average	.78	.78	.028	.027	.173	.172	.119	.108	.74	.74	.27	.26
	ADFM	.01	.00	.001	.001	.009	.006	.002	.003	.00	.01	.01	.01
M1-5E	Average	.67	.67	.012	.011	.130	.130	.073	.060	.77	.77	.28	.28
	ADFM	.01	.01	.001	.000	.006	.006	.001	.001	.00	.00	.01	.01
M6-10E	Average	.66	.66	.016	.016	.148	.147	.097	.090	.74	.75	.29	.28
	ADFM	.01	.00	.001	.001	.001	.001	.003	.003	.01	.00	.01	.00
M11-15E	Average	.78	.78	.028	.024	.175	.176	.114	.096	.74	.73	.27	.26
	ADFM	.01	.01	.001	.001	.010	.007	.001	.002	.00	.00	.00	.01
M1-5F	Average	.67	.65	.010	.011	.132	.133	.065	.070	.77	.77	.28	.28
	ADFM	.01	.02	.000	.001	.006	.006	.001	.005	.01	.01	.01	.01

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M6-10F	Average	.66	.68	.016	.018	.147	.151	.096	.097	.73	.74	.29	.28
	ADFM	.00	.03	.001	.003	.002	.006	.003	.003	.01	.01	.01	.01
M11-15F	Average	.78	.77	.026	.026	.177	.178	.104	.106	.74	.73	.26	.26
	ADFM	.02	.01	.002	.000	.007	.007	.002	.001	.00	.00	.00	.00

SUMMARY BY INGOT POSITION—Continued

M-A rails	Average	0.69	0.71	0.018	0.021	0.149	0.152	0.091	0.106	0.73	0.73	0.28	0.29
	ADFM	.01	.02	.001	.002	.009	.010	.005	.020	0	.01	.00	.00
	DFAR	.00	0	0	.000	0	0	0	0	0	0	0	0
M-B rails	Average	.71	.71	.020	.019	.156	.150	.099	.110	.75	.75	.28	.27
	ADFM	.01	.01	.001	.001	.010	.005	.002	.008	.01	.01	.01	.01
	DFAR	+ .02	0	+.002	+.002	+.007	−.002	+.008	+.004	+ .02	+ .02	.00	− .02
M-C rails	Average	.70	.70	.019	.019	.149	.148	.098	.096	.75	.75	.27	.28
	ADFM	.01	.01	.000	.001	.006	.006	.001	.002	.00	.01	.01	.01
	DFAR	+ .01	− .01	+.001	+.002	.000	−.004	+.007	−.010	+ .02	+ .02	− .01	− .01
M-D rails	Average	.71	.70	.019	.018	.148	.148	.099	.090	.75	.75	.28	.27
	ADFM	.01	.00	.001	.001	.006	.006	.002	.003	.00	.01	.01	.01
	DFAR	+ .02	− .01	+.001	−.003	−.001	−.004	+.008	−.016	+ .02	+ .02	0	− .02
M-E rails	Average	.70	.71	.019	.017	.150	.151	.095	.082	.75	.75	.28	.27
	ADFM	.01	.01	.001	.001	.005	.005	.002	.002	.00	.00	.01	.01
	DFAR	+ .01	.00	+.001	−.004	+.001	−.001	+.004	+.024	+ .02	+ .02	.00	− .02
M-F rails	Average	.70	.70	.017	.018	.152	.154	.088	.091	.74	.75	.28	.27
	ADFM	.01	.02	.001	.001	.005	.009	.002	.003	.01	.01	.01	.01
	DFAR	+ .01	− .01	−.001	−.003	+.003	+.002	−.003	−.015	+ .01	+ .02	.00	− .02

SUMMARY BY HEATS—Continued

M1-5	Average	0.67	0.67	0.012	0.012	0.129	0.130	0.071	0.076	0.77	0.77	0.28	0.28
	ADFM	.01	.01	.001	.001	.007	.007	.006	.012	.01	.01	.01	.01
M6-10	Average	.66	.65	.016	.017	.146	.147	.098	.103	.74	.74	.29	.29
	ADFM	.01	.02	.001	.001	.003	.004	.004	.010	.01	.02	.01	.01
M11-15	Average	.78	.78	.028	.027	.175	.175	.116	.114	.74	.74	.26	.27
	ADFM	.01	.01	.001	.001	.009	.009	.005	.002	.01	.01	.01	.01

GENERAL AVERAGE—Continued

	General average	0.70	0.71	0.019	0.019	0.151	0.151	0.095	0.096	0.75	0.75	0.28	0.28
	ADFM	.01	.001	.001	.001	.007	.007	.002	.006	.00	.01	.01	.01

[a] Average deviation from mean. [b] Deviation from A rail.

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(*d*) INSPECTION OF INGOTS.—The sink-head ingots were examined while cold for surface defects, and the surface condition of all ingots was noted by another observer as they passed through the blooming mill (Tables 4 and 5). There were side splashes, rough sides, and rough corners on nearly all the sink-head ingots, and only 4 of the 35 were noted as clean blooms in rolling, as compared with 8 clean blooms from the 15 comparison ingots.

(*e*) COMPARISON OF CHEMICAL COMPOSITION. (1) *Sink-Head Ingots* (Table 13).—The chemical composition of the rails from the sink-head ingots shows a remarkable degree of uniformity. Thus for the O analysis the average carbon is 0.648 per cent, with an average deviation from mean of 0.017. If we take the C test piece as representing the ingot composition, the average for carbon of C–M is 0.644 per cent and C–O 0.653 per cent, the average deviation from mean in each case being 0.017.

The average phosphorus content of the O position is 0.032 per cent, with an average deviation of 0.002; for sulphur (O position) the average is 0.047 per cent, with a deviation of 0.004. For the M position the concordance is somewhat less good. Manganese and silicon are very uniformly distributed.

(2) *Comparison Ingots* (Table 14).—These ingots were cast with small end uppermost, and, except for a nickel content of 0.75 per cent and chromium content of 0.28 per cent, were of a composition and uniformity comparable with the sink-head type, the average O position having a carbon content of 0.645 per cent (sink head 0.648 per cent), with an average deviation from mean of 0.036. The manganese content is 0.70 per cent, as compared with 0.89 per cent for the sink-head ingots; the phosphorus content is 0.019 per cent, while in the sink-head ingot it is 0.047 per cent. Group M11–M15 has an average phosphorus content of 0.028 per cent, which quite closely agrees with that of English-cast ingots. This subgroup is also from its method of manufacture the best to compare with the imported ingots.

The sulphur content of the comparison ingots follows the grouping by heats; the O analysis of C rail, for example, for M1–M5 has a sulphur content of 0.076 per cent; for M6–M10 the sulphur is 0.101 per cent; for M11–M15 the sulphur is 0.118 per cent. The manganese and silicon of the first two groups are practically identical, while both are slightly higher in the third group (M11–M15). The chromium and nickel contents are practically constant at 0.28 per cent for the former and 0.75 per cent for the latter for all three groups.

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4. THE SINK-HEAD INGOTS, THEIR MANUFACTURE AND CLASSIFICATION

There were 37 ingots of this Hadfield type cast large end up and each provided with a sink head (Table 1 and Fig. 4). The ingots were fed by the Hadfield method in the usual manner, with charcoal and blast continued for about 20 to 40 minutes until the molten steel had set on the top of the head.

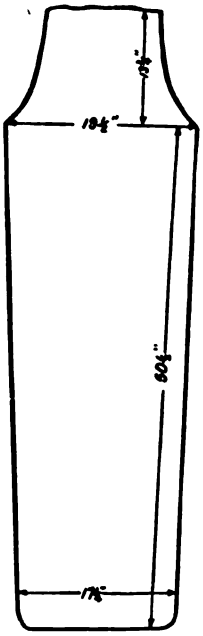

FIG. 4.—*Sink-head ingot*

The steel was made by the converter process, a small furnace being used, and each ingot represents a heat. These ingots are of remarkable uniformity of chemical composition, weight, and sink-head characteristics (Table 2). About 30 to 40 seconds were required for pouring each ingot.

(*a*) SINK-HEAD EFFECT.—Due to the uncertainty of the location of the sink-head junction, an arbitrary allowance has to be made in advance at hot saw or shears or one risks having the A rails rejected. If the total top discard (bloom and rail) is adjusted to 13 per cent there will be no rails rejected on account of sink head; if this is set at 8 per cent one would expect about one-half the A rails to contain surface imperfections caused by the sink head and ingot junction (Table 10). The sink head alone entails on the average a discard of about 9.1 per cent of the Hadfield ingot.

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The ingot usually enters the rolls with large end and sink head first; it appears, however, to make no difference which end of ingot is first subjected to the "squaring-up" pass. The presence of the sink head appears to present no difficulties in either the blooming mill or rail mill. One ingot cracked at the bottom of the sink head during rolling, but this was without effect on the passage of this bloom through the rolls.

(*b*) PIPING IN SINK-HEAD INGOTS.—Two of the 35 sink-head ingots were examined as rails had pipes, detected in the mill, extending into the ingot 10 or 11 per cent, or to some 5 per cent below the sink head in some cases (Table 10). All piping would appear to be eliminated with a discard of 11 per cent, which is very nearly that required to eliminate all sink-head imperfections and carbon segregation above 12 per cent.

(*c*) INFLUENCE OF SURFACE CONDITION OF INGOTS ON RAIL CLASSIFICATION.—It was possible to make a very careful examination for flaws in the surfaces of the sink-head ingots while cold; their condition during rolling, as well as of the comparison ingots, was also noted. In this way some idea of the relation of "second" rails to ingot surface may be established (Tables 4, 5, and 12).

There were a relatively large number of "second" rails from the sink-head ingots, 61 out of a total of 104 rails, or 59 per cent. There were only 4 in 57 from the comparison ingots, or 7 per cent. The A rails were not included for either group. This inspection was made by Pennsylvania Railroad Co. inspectors.

There appears to be no reason why ingots of the sink-head type should not be as well surfaced as any other, and the high percentage of seconds should not be attributed to the type of ingot, as the bottom of the sink head was well within the A rail, which was not classified. Probably the severe and unusual reheating conditions, together with the original surface as influenced by condition of the molds and teeming practice, contributed somewhat toward producing the large number of "seconds" from the sink-head ingots. A large percentage of "seconds" was similarly obtained from the reheated Hadfield ingots reported on by Hadfield and Burgess.[4]

5. THE COMPARISON INGOTS, THEIR MANUFACTURE AND CLASSIFICATION

There were 15 Maryland ingots, in 3 groups of 5 each, of about 7300 pounds each, top-poured into ingot molds with small end

[4] See footnote 1.

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up. (See Table 1 for dimensions.) They were rolled as portions of three heats on different dates, the total number of ingots in a heat being 13 or 20. The time of teeming into molds was about 35 seconds each. The product from each heat thus forms a distinctive group and should be considered separately. The ingots were selected arbitrarily before being cast as the five to be poured in succession after the second one of the heat. The heats were three of a series of rail steel from the ordinary output of the open hearth and were not especially chosen.

The first 10 ingots, heats 2×4497 (M1–M5) and 1×3632 (M6–M10) were apparently made from rising steel, while ingots M11–M15 of heat 2×4510 were from quiet or "killed" steel; ingots M6–M15 were deoxidized with aluminum; ingots M1–M10 were chilled on top, M1–M5 with cast-iron caps and M6–M10 with water.

The effects of these differences in furnace and casting practice are very striking on the finished product. Thus the deoxidized flat-top ingots (M11–M15) show, as would be expected, pronounced piping; for this group the average discard to physically sound steel is 26.5 per cent and ranging from 8.5 to 39.5 per cent. With this discard may be compared the discard of 5.2 per cent for the first Maryland group and 8 per cent for the second, and except in 2 cases out of 35 a negligible discard for piping below the sink head of the Hadfield ingots. The small discard (6.6 per cent) to sound steel for the average of the first two Maryland groups is, however, misleading, as there may exist internal or inclosed pipes, especially for the second or deoxidized group. In the first comparison group (M1–M5) one would expect to find a spongy, segregated structure in the upper part of the ingot. This is brought out in some of the sulphur prints of rail from the upper part of the ingots of this group, a typical one being shown in Fig. 8.

Of the first group, M5 contains a very pronounced pipe near the bottom of the ingot in the F position (Fig. 9). The question may be raised as to whether the practice of chilling the ingot top does not tend to produce internal pipes which may be distributed anywhere along the center line of the ingot.

The phenomenon of segregation, which is intimately allied to the furnace and casting practice for the comparison ingot, is discussed separately. The mechanical properties, as brought out by the several tests—ultimate strength, yield point, elongation,

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reduction of area, hardness and drop test—show less dependence on the processes of manufacture.

6. COMPARISON OF CHEMICAL SEGREGATION OF INGOTS OF BOTH TYPES

Each comparison ingot furnished 6 and each sink-head ingot 4 rails. In comparing the segregation of the A test piece it should be remembered that for the sink-head ingots this position is that of the drop-test piece at about 9 per cent below the top (including sink head), while for the comparison ingots the A test piece is at the junction of the X and A rails, or at 10.6 per cent from the top of ingot. Therefore they are in sufficiently similar positions for comparison, except that 10.6 per cent of a comparison ingot (7260 pounds) represents 770 pounds of steel, or 23 feet of rail, while 9.6 per cent of a sink-head ingot (5300 pounds) represents 509 pounds, or only 15 feet of rail, or the A position in the comparison has a rail discard 8 feet greater than the sink-head ingot on the average.

The E position of the rails from sink-head ingots should be compared with the position of the comparison ingots F, the sink head D with comparison E, the sink head C with comparison C and D, and the sink head B with comparison B.

For both types of ingot the longitudinal segregation of carbon is very slight along the O positions, while for the M positions, or along the center line of the ingot, it is quite marked, the carbon being greatest at A near the top of the ingot, decreasing to the H–D (or M–E) position, and rising again as the bottom of the ingot is approached. The results given are for the three types of comparison ingot, but for this effect, the type of ingot appears to be less important than for other factors.

The segregation of the sink-head ingots and of the three groups of ingots made with small end uppermost are discussed separately with reference to influence and method of manufacture.

(*a*) SEGREGATION IN SINK-HEAD INGOTS.—The sink-head ingots were of "piping" steel deoxidized with aluminum. Each represents a separate heat of converter steel.

These ingots were in general of remarkable uniformity as regards segregation (Table 15), thus showing the furnace and casting practice also to have been very uniform. At the A test piece, 9.1 per cent below the top of the ingot on the average, the segregation, though variable, is always positive; that is, the carbon content of the M position is greater than that of the O position. Fourteen of

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35 ingots have a segregation greater than 12 per cent and 3 greater than 25 per cent; for one of these (H13), the A position is only 5.1 per cent below the top of the ingot (Table 10).

For these 35 sink-head ingots the discard for segregation above 12 per cent is located 13 per cent (plus or minus 1.86) below top of ingot, with a maximum of 17.5 per cent and a minimum of 5.1 per cent (Tables 10 and 17).

TABLE 15.—Percentage Carbon Segregation in Rails from Sink-Head Ingots

Ingot	Rail A	Rail B	Rail C	Rail D	Rail E	Ingot	Rail A	Rail B	Rail C	Rail D	Rail E
H1	13.02	−0.15	−3.75	−4.59	1.23	H21	11.91	1.66	−0.75	0.45	2.61
H2	9.44	.00	−2.67	−6.77	−1.60	H22	2.67	− .75	.31	−3.48	− .30
H3	11.65	−5.33	−3.89	−7.42	− .79	H23	24.35	.77	−1.69	−4.09	−1.42
H4	16.49	4.83	−3.82	−3.41	− .62	H24	21.94	−2.26	−2.88	−2.93	−4.55
H5	.....	.....	−1.29	−1.30	− .44	H25	5.17	2.68	1.52	−2.56	2.73
H6	9.25	4.86	−1.73	−6.40	2.78	H26	34.16	.60	−1.22	−7.11	.92
H7	19.79	.30	−1.95	− .15	− .60	H27	4.76	1.38	.00	−4.48	3.09
H8	3.34	1.68	− .15	−5.20	1.57	H28	26.77	2.97	−3.12	−5.43	.16
H9	23.77	4.87	−1.12	−6.20	− .34	H29	10.50	3.58	−1.72	−7.55	.49
H10	20.63	2.17	.31	−4.54	3.09	H30	15.57	2.54	− .47	−4.32	− .97
H11	4.69	3.74	.32	−5.16	.32	H31	12.89	2.95	.31	−3.70	.62
H12	2.16	1.44	−2.41	−6.52	− .57	H33	3.83	2.06	−7.41	−2.68	.60
H13	46.69	−3.11	−4.35	−1.27	−2.79	H34	10.11	6.75	−2.72	−5.16	−2.96
H14	2.48	1.49	.00	−4.32	.30	H35	10.33	3.32	−5.11	−6.16	1.46
H15	7.83	4.08	−4.63	1.18	1.63	H36	15.48	2.37	2.54	−4.59	− .96
H16	13.93	4.52	−2.33	−9.74	−1.41	Average	+12.73	+2.32	−1.27	−4.48	+0.40
H17	1.50	4.75	1.22	−5.33	3.29	ADFM[a]	7.48	1.35	1.68	1.81	1.42
H18	5.20	1.96	− .30	−3.46	2.14						
H19	9.66	2.85	.00	−7.28	.81						
H20	1.04	2.60	−1.96	−5.07	1.73						

[a] Average deviation from mean.

TABLE 16.—Percentages Carbon Segregation in Rails from Comparison Ingots

Ingot	Rail A	Rail B	Rail C	Rail D	Rail E	Rail F
M1	30.88	7.17	−8.80	− 5.07	−12.34	−13.45
M2	30.39	27.55	.31	− 7.73	−13.06	3.93
M3	13.30	9.63	.96	− 5.79	−15.20	4.44
M4	20.17	5.28	−8.23	− 4.41	−15.89	2.96
M5	16.85	3.37	−1.25	− 7.61	−15.24	10.75
M6	16.51	9.20	−4.98	− 5.93	− 3.71	.62
M7	19.34	5.07	3.68	− 3.44	− 8.79	.00
M8	2.51	5.05	.91	− 5.76	− 6.65	.64
M9	9.15	5.38	.73	− 8.27	− 1.42	− 5.91
M10	.....	.....	− .31	− 3.73	− 4.41	.....
M11	3.08	− .87	−4.95	− 8.69	−11.78	− 1.16
M12	32.52	−1.61	−3.61	−11.00	−11.68	− 2.08
M13	.....	−1.98	−5.31	− 7.12	−13.82	− 1.47
M14	4.74	−6.26	−2.48	−11.92	− 8.62	.16
M15	.15	3.07	−3.73	−10.66	−11.09	2.61
Average	+14.41	+5.00	−2.42	−7.14	−10.40	+ .27
ADFM[a]	7.53	2.67	2.35	1.39	1.57	3.18

[a] Average deviation from mean.

At A the average segregation is +12.73 per cent, ±7.48; at B the segregation is +2.32, ±1.35, with only 4 negative values; at

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C it is −1.27 per cent, ±1.66, with 10 positive values; at D it is 4.48 per cent, ±1.81, with only 2 positive values; and at E only +0.40 per cent, ±1.42, with 15 negative values (Table 15).

(*b*) SEGREGATION IN COMPARISON INGOTS. (1) *Group M1–M5 (Heat 2×4497)*.—This heat has no aluminum addition in molds, and cast-iron caps were put on the tops of the ingots as soon as poured. It was found that the ingot tops were rounded after the steel had solidified; therefore, they were made of rising steel not specially deoxidized in the mold. The ingots were stripped 20 minutes after casting and remained in soaking pits from 2 hours and 10 minutes to 2 hours and 23 minutes. This is about as rapid handling of ingots in the molds as it is possible to have, although the time in pits could have been lessened. The resulting composition and segregation is strikingly characteristic of their history.

TABLE 17.—Necessary Total Discard (Percentage) in Rails from Sink-Head Ingots [a]

[H24 was omitted, as this ingot was given an excessive bottom crop due to error in judgment at mill. *s*=discard for segregation. *p*=discard for physical unsoundness.]

Ingot number	Top	Bottom (*p*)	Middle (*p*)	Chemical	Physical	Total
H1	*s* 16.2	3.0	1.8	16.2	4.8	21.0
H2	*s* 12.3	2.5	1.8	12.3	4.3	16.6
H3	*s* 13.9	3.4	2.6	13.9	6.0	19.9
H4	*s* 17.5	3.4	1.8	17.5	5.2	22.7
H6	*s* 13.4	3.7	1.9	12.4	5.6	18.0
H7	*s* 14.0	3.7	2.1	14.0	5.8	19.8
H8	*s* 12.1	3.9	2.4	12.1	6.3	18.4
H9	*s* 15.0	4.2	2.3	15.0	6.5	21.5
H10	*s* 15.6	3.6	2.0	15.6	5.6	21.2
H11	*s* 14.3	3.7	2.1	14.3	5.8	20.1
H12	*p* 10.7	4.1	2.1	.....	16.9	16.9
H13	*s* 13.0	3.8	1.8	13.0	5.6	18.6
H14	*s* 13.8	3.8	1.9	13.8	5.7	19.5
H15	*s* 12.6	3.3	2.2	12.6	5.5	18.1
H16	*s* 12.4	4.4	1.7	12.4	6.1	18.5
H17	*p* 9.1	4.0	2.4	.....	15.5	15.5
H18	*s* 13.9	3.9	2.4	13.9	6.3	20.2
H19	*s* 11.8	3.6	1.9	11.8	5.5	17.3
H20	*s* 14.0	3.7	2.1	14.0	5.8	19.8
H21	*s* 14.6	3.6	2.3	14.6	5.9	20.5
H22	*s* 11.5	3.4	1.8	11.5	5.2	16.7
H23	*s* 10.6	4.3	2.4	10.6	6.7	17.3
H25	*p* 9.4	4.2	2.3	.....	15.9	15.9
H26	*s* 14.3	3.8	1.9	14.3	5.7	20.0
H27	*s* 9.7	3.2	2.5	9.7	5.7	15.4
H28	*s* 13.8	2.7	2.0	15.8	4.7	8.5
H29	*s* 16.8	3.8	2.0	16.8	5.8	22.6
H30	*p* 9.8	4.3	2.3	.....	16.4	16.4
H31	*p* 8.8	3.0	1.6	.....	13.5	13.5
H33	*s* 12.5	4.6	2.4	12.5	7.0	19.5
H34	*s* 14.4	4.2	1.9	14.4	6.1	20.5
H35	*s* 11.6	4.0	1.8	11.6	5.8	17.4
H36	*s* 15.9	3.9	1.9	15.9	5.8	21.7
Average	13.0	3.7	2.1	11.5	7.2	18.4
Deviation	1.86	.....	.....	.....	.....	.....

[a] The total discard (top, middle, and bottom) includes the discard necessary to eliminate piping and segregation above 12 per cent.

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TABLE 18.—Necessary Total Discard (Percentage) in Rails from Comparison Ingots [a]

[*s*—discard for segregation. *p*—discard for physical unsoundness. B, E, D refer to rails having above 12 per cent segregation or having inclosed pipes.]

Ingot number	Top	Bottom	Middle	Chemical	Physical	Total
M1	*s* 31.9	*p* 4.7	*s* 32.0 E+D	63.9	4.7	68.6
M2	*s* 24.8	*p* 4.3	*s* 34.5 B+E+D	59.3	4.3	63.6
M3	*s* 23.9	*p* 3.1	*s* 34.4 E+D	58.3	3.1	61.4
M4	*s* 23.7	*p* 3.7	*s* 34.5 E+D	58.2	3.7	61.9
M5	*s* 28.6	*p* 3.3	*ps* 33.7 E+D	62.3	3.3	66.6
Average	26.6	3.4	33.8	60.4	3.8	64.2
M6	*s* 25.3	*p* 5.0	*p* 1.5	25.3	6.5	31.8
M7	*s* 23.7	*p* 3.9	*p* 1.3	23.7	5.2	28.9
M8	*s* 23.3	*p* 4.0	*p* 1.6	23.3	5.6	28.9
M9	*s* 23.8	*p* 4.8	*p* 1.1	23.8	5.9	29.7
Average	24.0	4.44	1.4	24.0	5.8	29.8
M11	*p* 29.0	*p* 5.1	*p* 1.6	0	35.7	35.7
M12	*s* 19.8	*p* 5.0	*p* 1.7	19.8	6.7	26.5
M13	*p* 39.5	*p* 4.1	[b] *p* 1.0	0	44.6	44.6
M14	*p* 28.2	*p* 4.6	[b] *p* 1.6	0	34.4	34.4
M15	*p* 27.4	*p* 4.8	[b] *p* 1.9	0	34.1	34.1
Average	26.8	4.7	1.6	4.0	31.1	35.1
Average	26.0	3.8	13.0	30.0	14.1	43.9
Deviation	±3.3	.....	.....	.....	.....	.....

[a] The total discard (top, middle, and bottom) includes the discard necessary to eliminate piping and segregation above 12 per cent.

[b] M_{13} and M_{14} contain trace of piping in B position. M_{15} has a small pipe at F.

The top of the ingot is characterized by high and variable segregation; thus the total top discard required to eliminate segregation above 12 per cent ranged from 23.7 to 31.9 per cent for the five ingots, as compared with a top discard of 2.6 to 9.3 per cent to the physically sound steel; that is, free from pipe (Tables 11 and 18). Within this range of discard for segregation, however, the segregation in a single ingot varied by enormous amounts, such as 26.52 to 94.62 per cent for ingot M2 and −7.36 to 49.28 per cent for ingot M5 (Table 20). This type of ingot is also characterized in all cases, by high negative segregation in the lower portion of the ingot, particularly marked at the E position, where it ranges from −12.34 per cent to −15.89 per cent (Table 16). Except for M1, at the bottom of the ingot F position (−13.45 per cent, Table 16), there is a reversal to relatively small positive segregation. All but M2 contain a well-marked soft streak at F, the bottom of the ingot, and in the case of M5 this streak incloses a pronounced pipe (Figs. 9 and 10). The total material that should be discarded from this type of ingot for segregation ranges from 58.2 to 63.9 per cent (Table 18). Provision should be made for a larger bottom-bloom crop than is customary to eliminate both segregation and piping in the bottom of the ingot. The bottom

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rail and the adjacent one from every ingot of this group (M1–M5) should be discarded for segregation. The bottom rail has in addition elements of physical weakness shown by the soft steel streaks which may inclose pipes.

(2) *Group M6–M10 (Heat 1×3632).*—The ingots of this heat had rounded tops on which water was put a few moments after casting; they were also treated while being cast with 2 ounces of aluminum per ton of steel. They were, therefore, of rising steel deoxidized in the molds, and should show somewhat greater tendency to piping than the first group; this is borne out by the results. The ingots of the second group required a discard averaging 8 per cent on account of pipe, while only one of the first group required a discard in excess of this amount (Table 11).

With respect to the B, C, D, E, and F positions for this group, the segregation is relatively low (max., 9.20 per cent, Table 16) and uniform from one ingot to another, as well as within any particular ingot, downward from the B position. The top discard for carbon segregation greater than 12 per cent (in O and M positions) is 23.3 to 25.3 per cent (Table 18), and within this discard the segregation is less extreme than in ingots of the first group, the maximum being about 25 per cent, which obtained in only 2 positions out of 29 (Table 20).

(3) *Group M11–M15 (Heat 2×4510).*—Two ounces of aluminum per ton were added to the ingots M11–M15 for the purpose of deoxidation. The ingot tops, which were flat after pouring into molds, were not chilled with caps or water. These ingots were, therefore, of quiet, piping steel and apparently more nearly approach the type of steel represented by the sink-head ingots than do the other two groups of comparison ingots. They remained somewhat longer in the molds (30 minutes as compared with 20 and 23 minutes) before stripping, and slightly longer in the soaking pits than the other comparison ingots.

The considerable piping at the top of the ingots of this group is very strikingly characteristic of the above open-hearth and casting practice. The carbon segregation in the upper half and at the bottom of this type of ingot is slight, but there is developed negative segregation ranging about 11 per cent at the D and E positions (Table 16). In contrast to the other two types of Maryland ingot the extent of the top discard is determined (Table 18), except for M12, which shows other anomalies, by piping (average discard 26.8 per cent) rather than by segregation (average discard 16.4 per cent, Table 22). The metallographic examination shows

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a small pipe in rail M15 at the bottom of the ingot in the F position, and incipient piping in M13 and M14 at the B position.

TABLE 19.—Percentage Segregation of Carbon in the "A" Rails of Sink-Head Ingots [a]

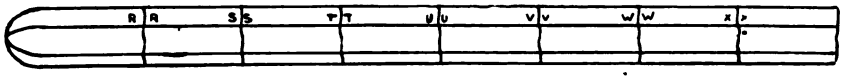

Diagram of cutting up of "A" rail of sink-head ingots

Ingot	Carbon	R	S	T	U	V	W	X
H1	Carbon O	0.632	0.649	0.647	0.650	0.658	0.648	0.647
	Carbon M	.902.	.789	.732	.683	.673	.674	.673
	Segregation	42.72	21.57	13.14	/ 5.08	2.28	4.01	4.02
H2	Carbon O	.610	.636	.635	.632	.634	.637	.636
	Carbon M	.967	.804	.705	.690	.659	.653	.654
	Segregation	58.52	26.42	/ 11.02	9.18	3.94	2.51	2.83
H3	Carbon O	.657	.670	.667	.653	.657	.....	.....
	Carbon M	.746	.708	.674	.682	.673	.....	.....
	Segregation	13.55	/ 5.67	1.05	4.44	2.44	.....	.....
H4	Carbon O	.605	.657	.654	.655	.649	.656	654
	Carbon M	.719	.777	.767	.708	.719	.691	.674
	Segregation	18.84	18.26	17.28	/ 8.09	10.79	5.34	3.06
H6	Carbon O	.640	.641	.641	.643	.644	.....	.....
	Carbon M	.805	.697	.675	.676	.674	.....	.....
	Segregation	25.78	/ 8.74	5.30	5.13	4.66	.....	.....
H7	Carbon O	.630	.669	.647	.667	.663	.662	.661
	Carbon M	.727	.889	.715	.686	.676	.671	.669
	Segregation	15.40	32.88	/ 10.51	2.85	1.96	1.36	1.21
H8	Carbon O	.649	.656	.657	.659	.654	.657	.....
	Carbon M	.758	.690	.673	.661	.662	.664	.....
	Segregation	16.80	/ 5.18	2.44	.30	1.22	1.07	.....
H9	Carbon O	.655	.662	.642	.639	.640	.649	.....
	Carbon M	.804	.742	.686	.658	.662	.674	.....
	Segregation	22.75	12.08	/ 6.85	2.97	3.44	3.85	.....
H10	Carbon O	.661	.638	.638	.643	.653	.643	.....
	Carbon M	.727	.717	.669	.667	.667	.660	.....
	Segregation	9.98	12.38	/ 4.86	3.73	2.14	2.64	.....
H11	Carbon O	.639	.643	.638	.636	.638	.620	.....
	Carbon M	.733	.674	.637	.653	.659	.655	.....
	Segregation	14.71	/ 4.82	.16	2.67	3.29	5.65	.....
H12	Carbon O	.700	.701	.695	.699	.712	.....	.....
	Carbon M	.743	.713	.712	.707	.702	.....	.....
	Segregation	16.14	1.71	2.45	1.14	−1.40	.....	.....
H13	Carbon O	.627	.645	.649	.657	.642	.651	.656
	Carbon M	.786	.739	.664	.667	.658	.666	.663
	Segregation	25.36	14.57	/ 2.31	1.52	2.49	2.30	1.07
H14	Carbon O	.655	.676	.692	.687	.683	.692	.682
	Carbon M	.839	.775	.711	.703	.693	.698	.696
	Segregation	28.09	14.64	/ 2.75	2.33	1.46	.87	2.05
H15	Carbon O	.691	.692	.696	.693	.689	.684	.....
	Carbon M	.758	.819	.761	.725	.719	.713	.....
	Segregation	9.70	18.35	/ 9.34	4.62	4.35	4.24	.....
H16	Carbon O	.685	.687	.693	.674	.683	.....	.....
	Carbon M	.776	.731	.689	.706	.715	.....	.....
	Segregation	13.28	/ 6.40	− .58	4.75	4.69	.....	.....
H17	Carbon O	.667	.675	.669	.657	.659	.659	.....
	Carbon M	.699	.705	.686	.688	.670	.687	.....
	Segregation	/ 4.80	4.44	2.54	4.72	1.67	4.25	.....

[a] The rails were broken at about 5-foot intervals for analysis in accordance with diagram to determine the top discard necessary to eliminate carbon segregation above 12 per cent. The point at which the segregation becomes less than this amount is indicated by "/" in the table.

[b] H25 ingot — Carbon O 0.590; Carbon M .702; Segregation 18.98

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TABLE 19—Percentage Segregation of Carbon in the "A" Rails of Sink-Head Ingots—Continued

Ingot	Carbon	R	S	T	U	V	W	X
H18	Carbon O	0.672	0.672	0.672	0.662	0.676	.....	.....
	Carbon M	.771	.772	.712	.692	.669	.....	.....
	Segregation	14.73	14.88	/ 5.95	4.53	−1.04	.....	.....
H19	Carbon O	.634	.631	.635	.631	.632	.....	.....
	Carbon M	.732	.701	.655	.654	.656	.....	.....
	Segregation	15.46	/ 11.09	3.15	3.65	3.80	.....	.....
H20	Carbon O	.638	.661	.663	.660	.668	.....	.....
	Carbon M	.752	.686	.689	.684	.679	.....	.....
	Segregation	17.87	/ 3.78	3.92	3.64	1.65	.....	.....
H21	Carbon O	.676	.674	.680	.668	.679	.....	.....
	Carbon M	.809	.707	.708	.692	.697	.....	.....
	Segregation	19.67	/ 4.90	4.12	3.59	2.65	.....	.....
H22	Carbon O	.678	.671	.671	.674	.681	.....	.....
	Carbon M	.725	.680	.682	.688	.707	.....	.....
	Segregation	/ 6.93	1.34	1.64	2.08	3.82	.....	.....
H23	Carbon O	.644	.655	.646	.647	.644	.....	.....
	Carbon M	.790	.703	.673	.669	.665	.....	.....
	Segregation	22.67	/ 7.33	4.18	3.40	3.26	.....	.....
H24	Carbon O	.618	.628	.629	.626	.....	.....	.....
	Carbon M	.835	.791	.698	.657	.....	.....	.....
	Segregation	35.11	25.96	/ 10.97	4.95	.....	.....	.....
H25	Carbon O	.596	.596	.599	.599	.591	.....	.....
	Carbon M	.633	.627	.625	.609	.609	.....	.....
	Segregation	/ 6.21	5.20	4.34	1.67	3.05	.....	.....
H26	Carbon O	.657	.665	.658	.665	.660	0.661	.....
	Carbon M	.858	.760	.714	.685	.678	.689	.....
	Segregation	30.59	14.29	/ 8.51	3.01	2.73	4.24	.....
H27	Carbon O	.655	.666	.654	.661	.650	.643	.....
	Carbon M	.722	.707	.662	.669	.656	.666	.....
	Segergation	/ 10.23	6.16	1.22	1.21	.92	3.58	.....
H28	Carbon O	.642	.645	.657	.647	.649	.647	.....
	Carbon M	.749	.746	.689	.673	.665	.666	.....
	Segregation	16.67	15.66	/ 4.87	4.02	2.47	2.94	.....
H29	Carbon O	.650	.646	.638	.654	.644	.....	.....
	Carbon M	.754	.682	.667	.671	.668	.....	.....
	Segergation	16.00	/ 5.57	4.55	2.60	3.73	.....	.....
H30	Carbon O	.626	.637	.631	.636	.635	.....	.....
	Carbon M	.681	.666	.662	.661	.656	.....	.....
	Segregation	/ 8.79	4.55	4.91	3.93	3.31	.....	.....
H31	Carbon O	.646	.643	.647	.655	.658	.650	.....
	Carbon M	.717	.679	.657	.646	.668	.665	.....
	Segregation	/ 10.99	5.60	1.55	−1.37	1.52	2.31	.....
H33	Carbon O	.776	.675	.679	.672	.678	.....	.....
	Carbon M	.800	.729	.702	.706	.710	.....	.....
	Segregation	18.34	/ 8.00	3.39	5.06	4.72	.....	.....
H34	Carbon O	.659	.655	.661	.665	.656	.....	.....
	Carbon M	.823	.725	.724	.723	.707	.....	.....
	Segregation	24.89	/ 10.69	9.53	8.72	7.77	.....	.....
H35	Carbon O	.622	.633	.639	.639	.625	.633	.....
	Carbon M	.842	.684	.677	.663	.646	.647	.....
	Segregation	35.37	/ 8.06	5.95	3.76	3.36	2.21	.....
H36	Carbon O	.634	.642	.654	.640	.648	.637	.....
	Carbon M	.796	.722	.655	.666	.649	.655	.....
	Segregation	25.55	12.46	/ .15	4.06	.15	2.83	.....

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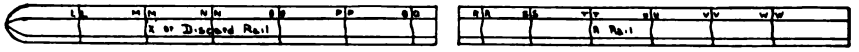
TABLE 20.—Percentage Segregation of Carbon in the "A" and Discard Rails of Comparison Ingots[a]



Ingot	Carbon	L	M	N	O	P	Q	R	S	T	U	V	W
M1	Carbon O	0.488	0.564	0.554	0.573	0.579	0.588	0.616	0.637	0.629	0.638	0.631	0.629
	Carbon M	.619	.630	.731	.848	.808	.791	.750	.740	.707	.683	.717	.662
	Segregation	26.84	11.70	31.95	47.99	39.55	34.52	21.75	16.17	12.40	7.05	13.63	/ 5.25
M2	Carbon O	.511	.558	.602	.622	.630	.644	.619	.641	.....	.....	.....	.....
	Carbon M	.886	1.086	.915	.898	.894	.878	.849	.811	.....	.....	.....	.....
	Segregation	73.39	94.62	51.99	44.37	41.90	36.34	37.16	26.52	/.....	.....	.....	.....
M3	Carbon O	.600	.625	.635	.665	.663	.616	.667	.676	.673	.....	.....	.....
	Carbon M	.774	.988	.889	.847	.810	.818	.792	.756	.708	.....	.....	.....
	Segregation	29.00	58.08	40.00	27.37	22.17	32.79	18.74	/ 11.83	5.20	.....	.....	.....
M4	Carbon O	.496	.630	.555	.626	.643	.664	.671	.659	.647	.....	.....	.....
	Carbon M	.661	.704	.884	.900	.861	.808	.752	.669	.690	.....	.....	.....
	Segregation	33.27	11.75	59.28	43.77	33.90	21.69	12.07	/ 1.52	6.65	.....	.....	.....
M5	Carbon O	.589	.605	.621	.626	.652	.630	.647	.652	.658	.641	.....	.....
	Carbon M	.672	.777	.927	.919	.844	.786	.761	.700	.734	.721	.....	.....
	Segregation	14.09	28.43	49.28	46.81	29.45	24.76	17.62	7.36	11.55	12.48	/.....	.....
M6	Carbon O	.618	.648	.662	.666	.677	.674	.684	.683	.690	.684	.687	.....
	Carbon M	.685	.701	.754	.783	.807	.769	.766	.769	.727	.764	.727	.....
	Segregation	10.84	8.18	13.90	17.57	19.20	14.09	11.99	12.59	/ 5.36	11.70	5.82	.....
M7	Carbon O	.640	.651	.662	.654	.666	.679	.678	.685	.689	.686	.677	.....
	Carbon M	.670	.718	.769	.799	.788	.777	.764	.729	.719	.714	.704	.....
	Segregation	4.69	10.29	16.16	22.17	18.32	14.43	12.68	/ 6.42	4.35	4.08	3.99	.....
M8	Carbon O	.585	.666	.655	.657	.662	.667	.667	.697	.691	.691	.687	.....
	Carbon M	.690	.667	.793	.805	.838	.797	.757	.732	.754	.727	.699	.....
	Segregation	17.95	0.15	21.07	22.53	26.59	19.49	13.49	/ 5.02	9.12	5.21	1.75	.....
M9	Carbon O	.626	.654	.671	.656	.700	.688	.699	.705	.706	.706	.703	.....
	Carbon M	.545	.685	.746	.839	.841	.818	.800	.786	.764	.743	.735	.....
	Segregation	−12.94	4.74	11.18	27.90	20.14	18.90	14.45	/ 11.49	8.22	5.24	4.55	.....

[a] The rails were broken at about 5-foot intervals for analysis in accordance with diagram to determine the top discard necessary to eliminate carbon segregation above 12 per cent. The point at which the segregation becomes less than this amount is indicated by "/" in the table.

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TABLE 20.—Percentage Segregation of Carbon in the "A" and Discard Rails of Comparison Ingots—Continued

Ingots	Carbon	L	M	N	O	P	Q	R	S	T	U	V	W
M11	Carbon O	0.703	0.723	0.724	0.712	0.701	0.717	0.705	0.688	0.693	0.689	.....	.....
	Carbon M	.770	.799	.770	.728	.758	.708	.698	.661	.669	.701	.....	.....
	Segregation	/ 9.53	10.51	6.35	2.25	8.13	− 1.26	−0.99	−3.92	−3.46	1.74	.....	.....
M12	Carbon O	.620	.671	.670	.701	.710	.699	.698	.692	.696	.680	.....	.....
	Carbon M	.816	.830	.871	.887	.806	.778	.745	.713	.672	.686	.....	.....
	Segregation	31.61	23.70	30.00	26.53	13.52	/ 11.30	6.73	3.03	−3.45	0.88	.....	.....
M13	Carbon O	.697	.694	.711	.712	.716	.713	.720	.707	.724	.709	.....	.....
	Carbon M	.765	.776	.788	.750	.778	.765	.688	.707	.653	.640	.....	.....
	Segregation	/ 9.76	11.82	10.83	5.34	8.66	7.29	−4.44	0.00	−9.81	−9.73	.....	.....
M14	Carbon O	.714	.711	.694	.700	.692	.705	.705	.706	.708	.682	0.701	.....
	Carbon M	.684	.742	.756	.781	.783	.776	.746	.736	.705	.679	.637	.....
	Segregation	−4.20	4.36	8.93	11.57	13.15	/ 10.07	5.82	4.25	−0.42	−0.44	−9.13	.....
M15	Carbon O	.695	.690	.705	.693	.701	.692	.685	.705	.712	.716	.704	0.663
	Carbon M	.664	.816	.797	.770	.815	.780	.722	.690	.696	.671	.677	.706
	Segregation	−4.46	18.26	13.05	11.11	16.26	12.72	/ 5.40	−2.13	−2.25	−6.28	−3.84	6.49

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A striking characteristic of this group (M11–M15) is the freedom from segregation in the A position; excluding M12 A for which the segregation is 32.52 per cent, the average for the others is about 2 per cent segregation at the A position (Table 16). M12 also possesses the following irregular segregation characteristics: At A there is segregation to such an extent that the steel is hypereutectoid; at B, a narrow hypereutectoid streak exists: at C there is the same structure as at A; and again at D a heavy hypoeutectoid streak.

(*c*) SEGREGATION OF ELEMENTS OTHER THAN CARBON.—In general, it might be expected that the elements sulphur and phosphorus would follow carbon in segregation characteristics, and that manganese, silicon, chromium, and nickel would show little or no segregation. These expectations are borne out in both the sink-head and comparison ingots.

Longitudinal segregation of phosphorus is absent for the O position and slight for the M position. There is practically no longitudinal or transverse sulphur segregation in the sink-head ingots, while for the comparison ingots the results on sulphur segregation were erratic but on the whole follow that of the carbon. For the elements manganese, nickel, chromium, and silicon there is no well-defined segregation within the ingot.

7. MECHANICAL TESTS

The mechanical tests included the ordinary drop test made on the first 5-foot piece below the top of the ingot showing physically sound steel. For the Hadfield rails this location was, in all but two cases, determined by the junction of sink head with the ingot proper. The 5-foot pieces formed by cutting up the A rails (also the X rail from the comparison ingots), as well as the test pieces, B, C, D, E (and F for the comparison ingots), were examined for hardness (Brinell and scleroscope) and behavior when stressed in tension.

(*a*) METHOD OF EXPRESSING THE RESULTS OF MECHANICAL TESTS.—It was thought best to express measurements, such as yield point, ultimate tensile strength, elongation, etc., in terms of the average and average deviation from the mean.[5]

When using the usual method of expressing results of this kind in terms of maximum and minimum values, undue emphasis is placed upon the occasional widely divergent figure, while the very

[5] The deviation of a single observation is the amount by which that observation departs from the mean or average value. By subtracting each observation from the mean without regard to sign, and averaging these, the average deviation from the mean is obtained.

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close agreement that may exist is not brought to light. Graphic plots of the various physical properties of both kinds of steel will be found in Figs. 13 to 17, inclusive.

TABLE 21.—Total Top Discard Required in Sink-Head Ingots[a]

Ingot	Weight of ingot	Bloom crop	Rail crop	Number of 5-foot pieces of rail	Length of cut	Amount of rail to cut	Rail discard, per cent of ingot	Bloom crop and rail discard	Total top discard required, per cent of ingot
	Pounds	Pounds	Feet		Feet	Pounds		Pounds	
H1	5300	190	5	3	20	667	12.6	857	16.2
H2	5300	155	5	2	15	500	9.4	655	12.3
H3	5300	205	11	1	16	533	10.1	738	13.9
H4	5350	270	5	3	20	667	12.5	937	17.5
H6	5300	324	5	1	10	333	6.3	657	12.4
H7	5250	234	5	2	15	500	9.5	734	14.0
H8	5300	142	10	1	15	500	9.4	642	12.1
H9	5300	294	5	2	15	500	9.4	794	15.0
H10	5250	320	5	2	15	500	9.5	820	15.6
H11	5300	325	8	1	13	433	8.2	758	14.3
H12	5350	204	10	0	10	333	6.2	537	10.0
H13	5300	191	5	2	15	500	9.4	691	13.0
H14	5300	231	5	2	15	500	9.4	731	13.8
H15	5300	167	5	2	15	500	9.4	667	12.6
H16	5350	199	9	1	14	467	8.7	666	12.4
H17	5300	145	10	0	10	333	6.3	478	9.0
H18	5300	237	5	2	15	500	9.4	737	13.9
H19	5300	160	9	1	14	467	8.8	627	11.8
H20	5300	276	9	1	14	467	8.8	743	14.0
H21	5300	442	5	1	10	333	6.3	775	14.6
H22	5250	437	5	0	5	167	3.2	604	11.5
H23	5350	134	8	1	13	433	8.1	567	10.6
H24	5300	185	9	2	19	633	11.9	818	15.4
H25	5300	160	10	0	10	333	6.3	493	9.3
H26	5300	160	8	2	18	600	11.3	760	14.3
H27	5300	245	8	0	8	267	5.0	512	9.7
H28	5300	233	5	2	15	500	9.4	733	13.8
H29	5350	566	5	1	10	333	6.2	899	16.8
H30	5300	247	8	0	8	267	5.0	514	9.7
H31	5300	234	7	0	7	233	4.4	467	8.8
H33	5300	228	8	1	13	433	8.2	661	12.5
H34	5300	195	12	1	17	567	10.7	762	14.4
H35	5300	280	5	1	10	333	6.3	613	11.6
H36	5300	342	5	2	15	500	9.4	842	15.9

[a] The above discard includes amount necessary to eliminate piping and segregation above 12 per cent

TABLE 22.—Total Top Discard Required in Comparison Ingots[a]

Ingot	Weight of ingot	Bloom crop	Rail crop	Number of 5-foot pieces of rail	Length of cut	Amount of rail to cut	Rail discard, per cent of ingot	Bloom crop and rail discard	Total top discard required, per cent of ingot
	Pounds	Pounds	Feet		Feet	Pounds		Pounds	
M1	7360	183	5	12	65	2167	29.4	2350	31.9
M2	7030	243	5	8	45	1500	21.3	1743	24.8
M3	7040	185	5	8	45	1500	21.3	1685	23.9
M4	7130	189	5	8	45	1500	21.0	1689	23.7
M5	7190	223	5	10	55	1833	25.5	2056	28.6
M6	7340	189	5	9	50	1667	22.7	1856	25.3
M7	7200	205	5	8	45	1500	20.8	1705	23.7
M8	7260	188	5	8	45	1500	20.7	1688	23.3
M9	7240	225	5	8	45	1500	20.7	1725	23.8
M11	7540	664	5	0	5	167	2.2	831	11.1
M12	7340	288	5	6	35	1167	15.9	1455	19.8
M13	7190	490	5	0	5	167	2.3	657	9.1
M14	7320	379	5	6	35	1167	15.9	1546	21.1
M15	7480	236	5	7	40	1333	17.8	1569	21.0

[a] The above discard includes amount necessary to eliminate piping and segregation above 12 per cent.

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(*b*) ULTIMATE STRENGTH AND YIELD POINT.—The ultimate strength of the steel in rails from both types of ingots are of the same order of magnitude, about 125 000 pounds per square inch.

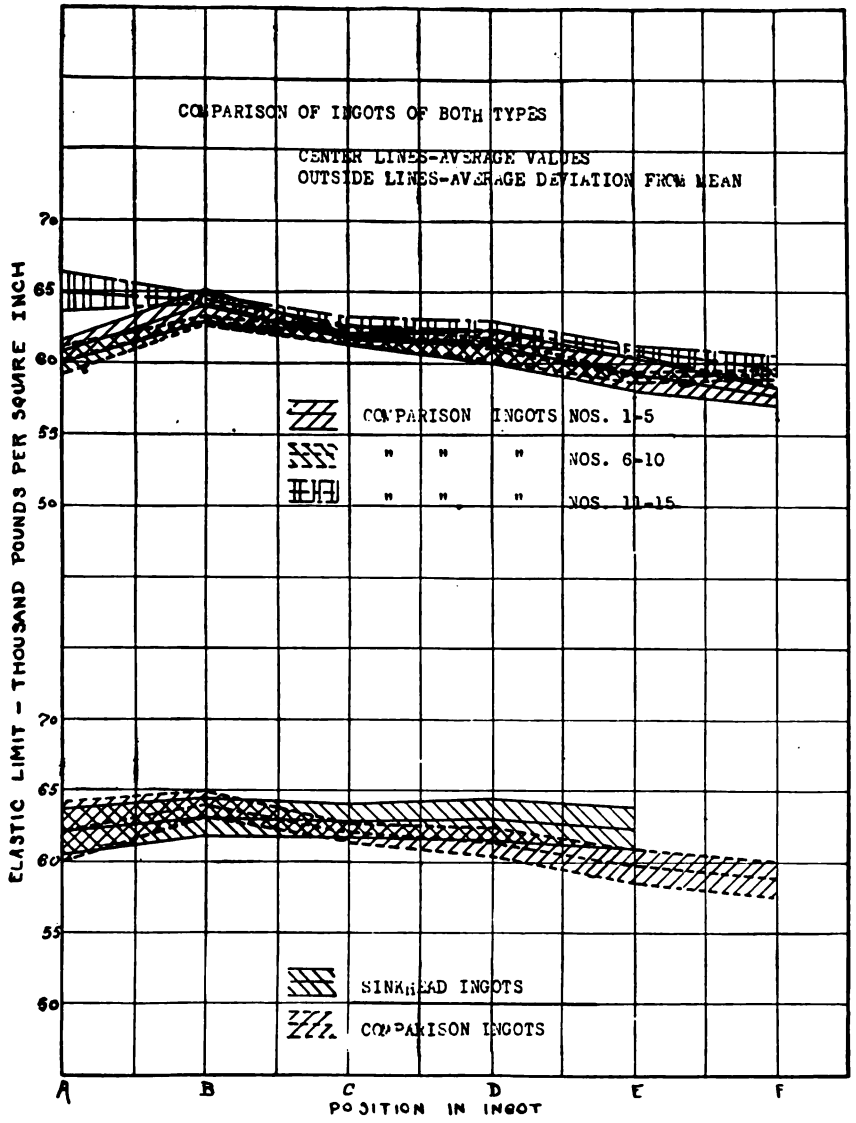

FIG. 13.—*Elastic limit of ingots of both types*

For the 35 sink-head ingots the nonuniformity, expressed as the average deviation from the mean for values at the A, B, C, D, and E positions, is somewhat greater than for the comparison ingots, except for the A position, at which the results are erratic (Tables 13 and 14).

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If we consider the ultimate strength throughout the length of the ingot, we note for the sink-head ingots there is practical uniformity down to the E position; for the comparison ingots, on the

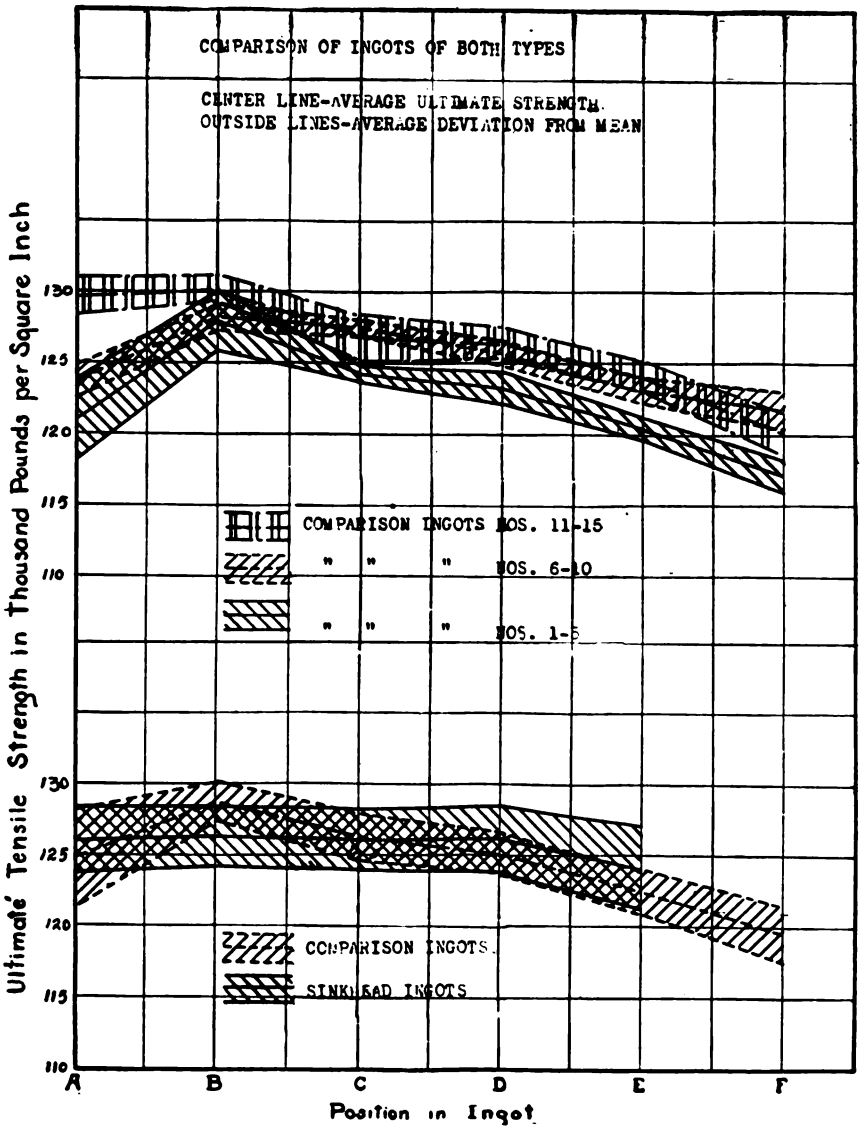

FIG. 14.—*Ultimate strength of ingots of both types*

other hand, there is an abrupt increase in ultimate strength from the A to the B position and a gradual falling off until the bottom of the ingot is reached, so that at the F position the ultimate tensile strength is some 9000 pounds per square inch less than the maximum at B.

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If the comparison ingots are considered by groups corresponding to heats, it is seen that the ultimate strength of each group may be distinguished from the others. Thus groups M1–M5 and M6–M10

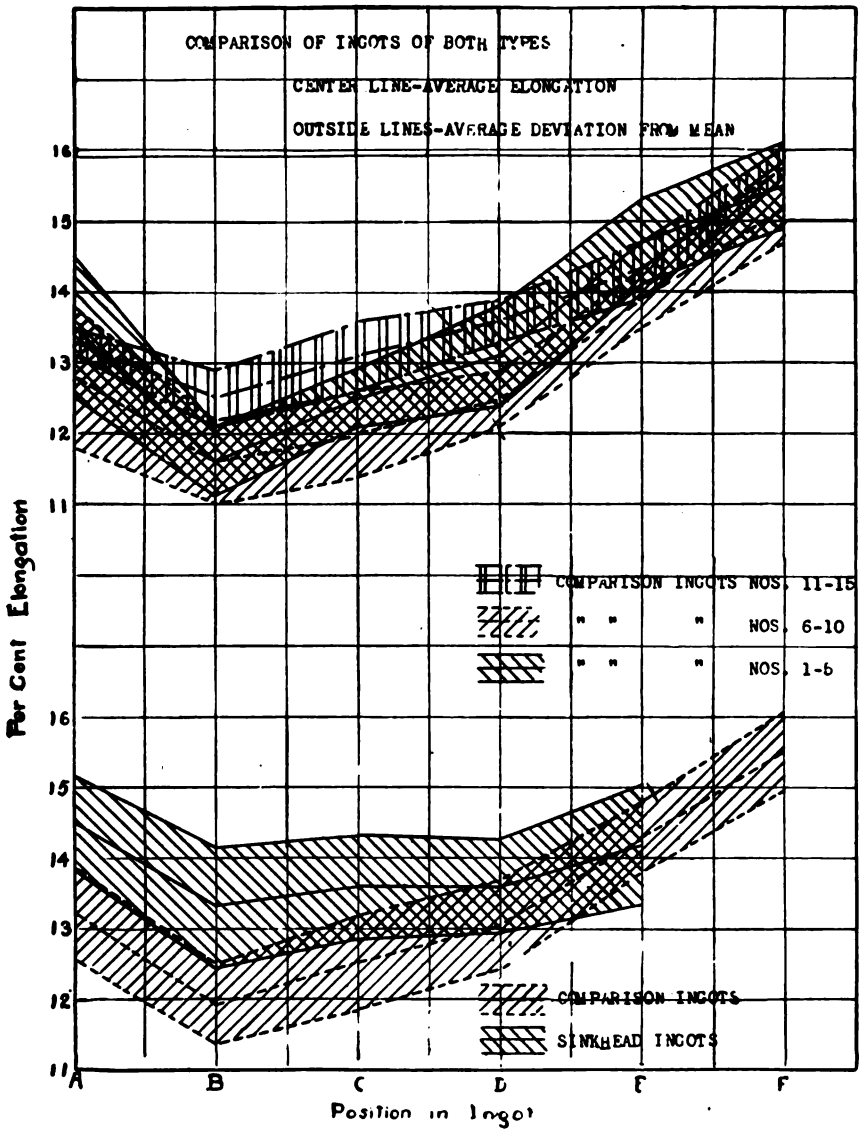

FIG. 15.—*Elongation of ingots of both types*

run closely parallel through the ingot, with M6–M10 having slightly higher ultimate strength. Group M11–M15 is characterized by the abnormally high value (approximately 130 000 pounds per square inch) in the A and B positions.

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The yield point shows, in general, characteristics similar to the ultimate strength; for the former, however, there is a somewhat greater uniformity throughout the ingot for all types, and also for

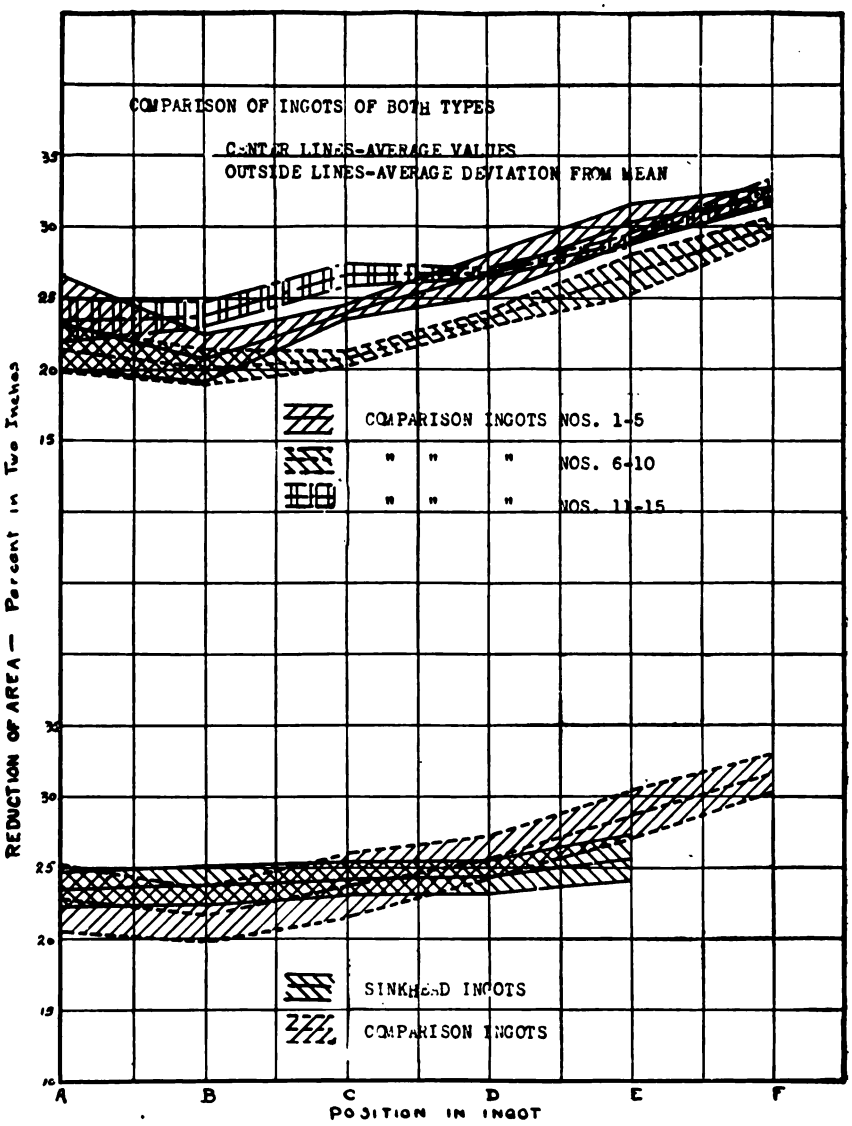

FIG. 16.—*Reduction of area of ingots of both types*

the ingots of a given group. The sink-head ingots have a slightly higher yield point (62 370 pounds per square inch) than the comparison ingots (61 370 pounds per square inch). For the three groups of the latter there is a falling off throughout the length of the ingot from approximately 64 000 pounds to 59 000 pounds per

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square inch (as for the ultimate strength), while for the sink-head ingots the yield point remains practically constant throughout.

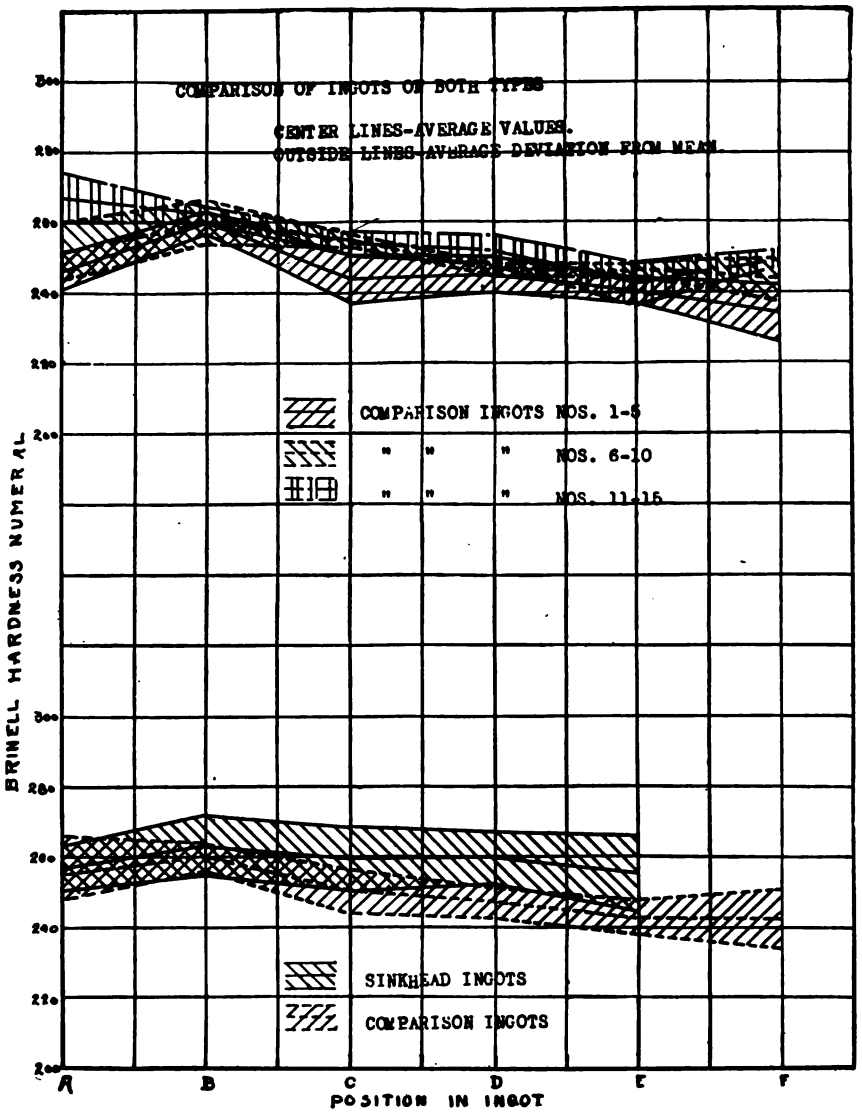

FIG. 17.—*Brinell hardness of ingots of both types*

(*c*) ELONGATION AND REDUCTION OF AREA.—The distribution of ductility, as measured by elongation in tension-test pieces from rails through the length of the sink-head ingots, is somewhat more uniform than for the comparison ingots. The elongation curves are practically the reciprocal of the tensile-strength curves.

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The average elongation of the rails from the sink-head ingots is 13.8 per cent, of the comparison ingots 13.6 per cent; the reduction of area for the former type of ingot is 24.38 per cent, for the latter type it is 25.78 per cent, with a range of 21.73 per cent at B to 31.68 per cent at the F position. There are also considerable differences among the three groups of comparison ingots.

(*d*) HARDNESS.—The scleroscope hardness numbers do not appear to have any significance in defining any differences in qualities or properties in the steel of the several ingots. The material from the sink-head ingots have an average hardness of 31, while the steel from ingots cast in the usual manner has an average hardness of 32.

The Brinell numerals are, as would be expected, fairly closely proportional to the ultimate strengths, and they appear to follow even more exactly the yield points. Whether or no there is any definite relation between carbon segregation and Brinell numerals appears difficult to say. The holes for obtaining drillings for the former are ½ inch in diameter and not always exactly coincident in location with the Brinell impression. In a general way one would expect the two to bear some relation to each other. The rails from the sink-head ingots had an average Brinell hardness of 260 (deviation ±8), while the rails made from ingots cast with the small end uppermost without sink heads had a Brinell hardness of 250 (deviation ±6).

(*e*) DROP TESTS.—In the drop tests blows were given on head of rail to destruction, the deflection was measured for each blow, the elongation for each inch over 6 inches and the total elongation noted.

The drop-test piece was taken from near the top of the ingot, and in each case was the first 5-foot length of rail immediately after reaching physically sound steel; that is, free from pipe and in the case of the rails from sink-head ingots also below the bottom of the sink head. Of the 14 comparison rails none broke under 4 or 5 blows (6 rails were nicked and broken after the fourth or fifth blow), while for the 35 Hadfield rails the range was 2 to 7 blows, there being 9 requiring 4 blows, and 4 requiring 3 blows; only 22, or 63 per cent, of the rails from sink-head ingots withstood 4 or more blows. The ductility as measured by the deflection and elongation is also somewhat greater and more uniform for the comparison rails.

The greater uniformity of the sink-head over the comparison ingots, as shown in nearly all other mechanical tests,

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FIG. 5.—*Photograph of sulphur print of top bloom from comparison ingot No. 10*

FIG. 6.—*Photograph of sulphur print of top bloom from sink-head ingot No. 5*

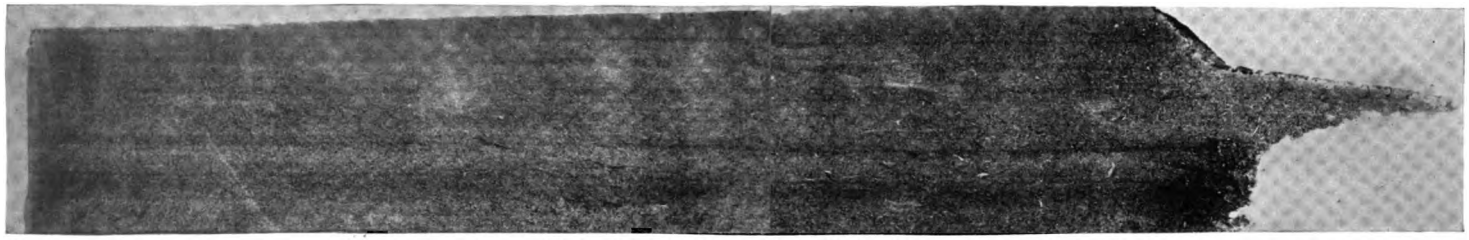

FIG. 7.—*Photograph of sulphur print of half section of sink-head ingot No. 32*

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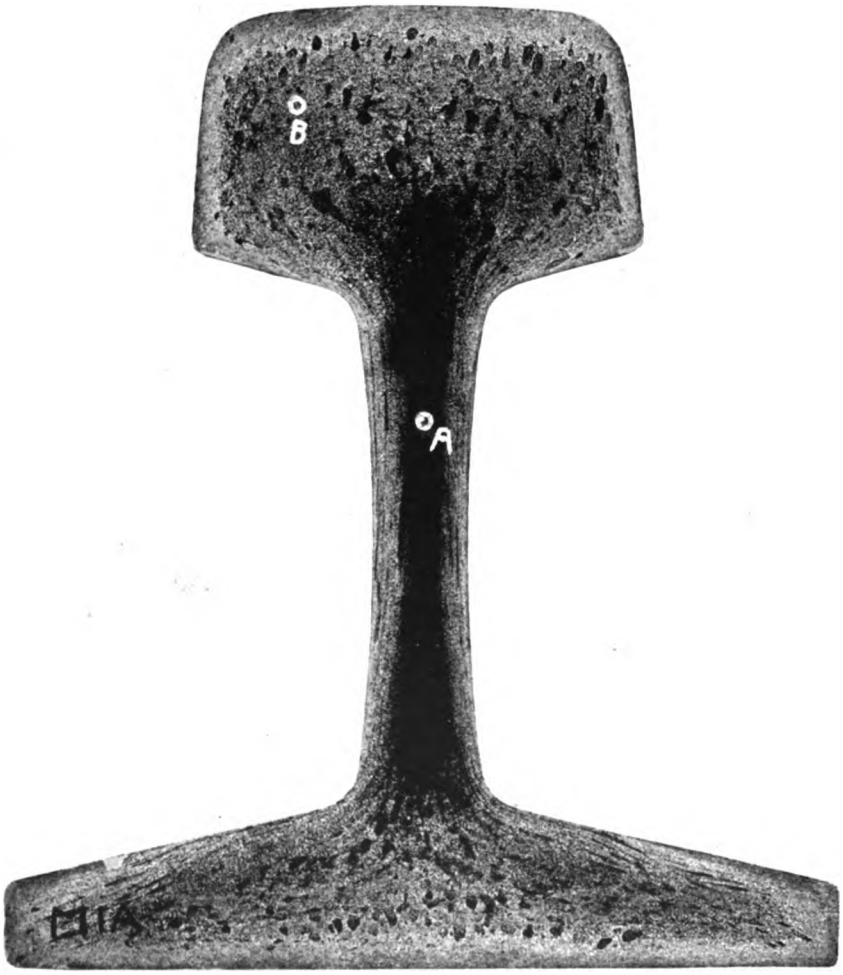

FIG. 8.—*Rail M1–A from comparison ingot*

Sulphur print, showing extreme segregation in rail from top of ingot. × 1

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a

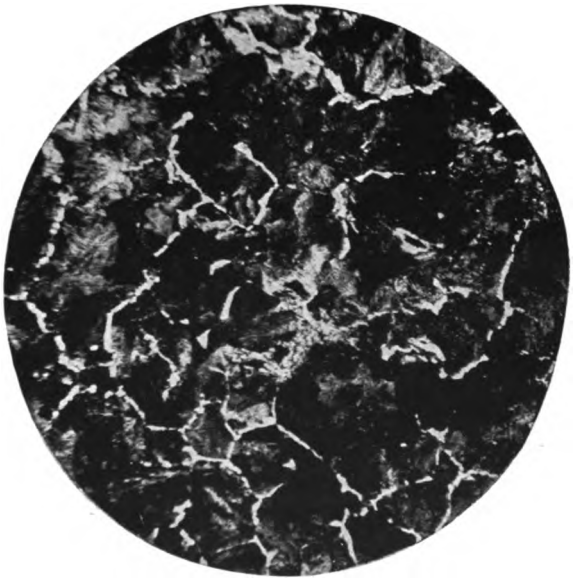

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FIG. 8.—*Rail M1–A from comparison ingot (continued)*

Typical microstructure of the metal of rail M1-A at positions indicated in sulphur print. × 50, reduced from × 100

(*a*) Moderately finely grained hypereutectoid steel, pearlite with a trace of free cementite and many small slag inclusions

(*b*) Moderately finely grained hypoeutectoid steel, pearlite with a thick irregular ferrite network

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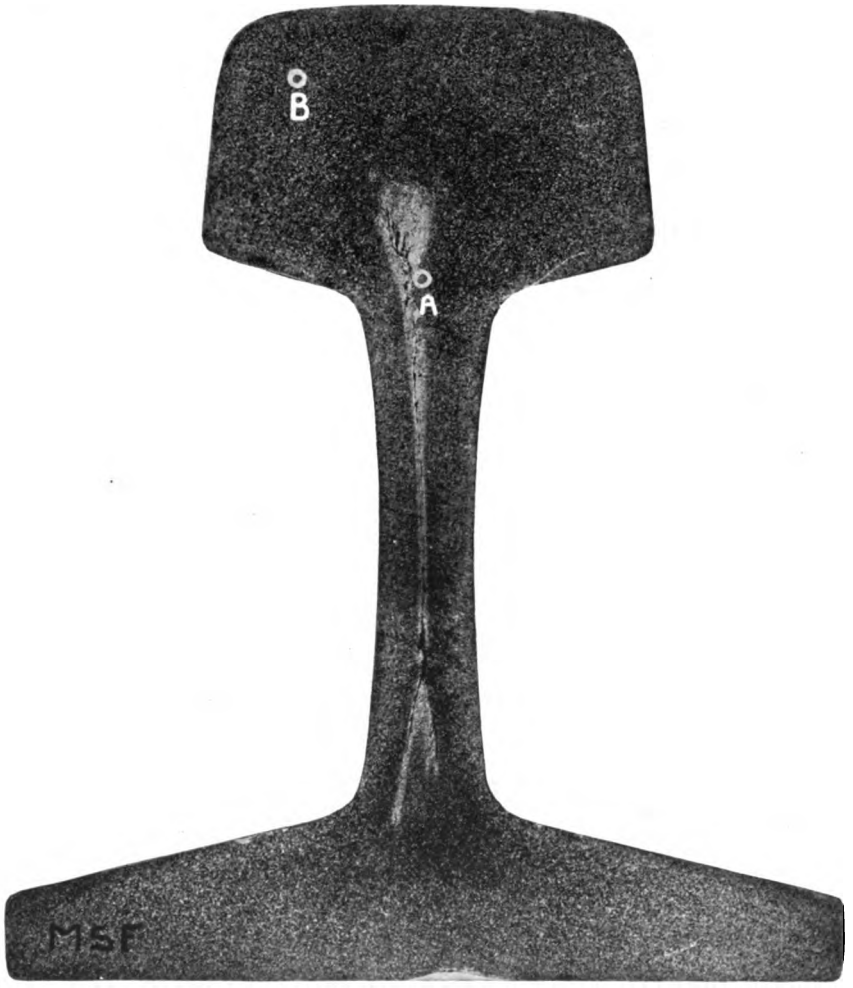

FIG. 9.—*Rail M5–F from comparison ingot*

Sulphur print, showing pipe in rail from bottom portion of ingot (F position). × 1

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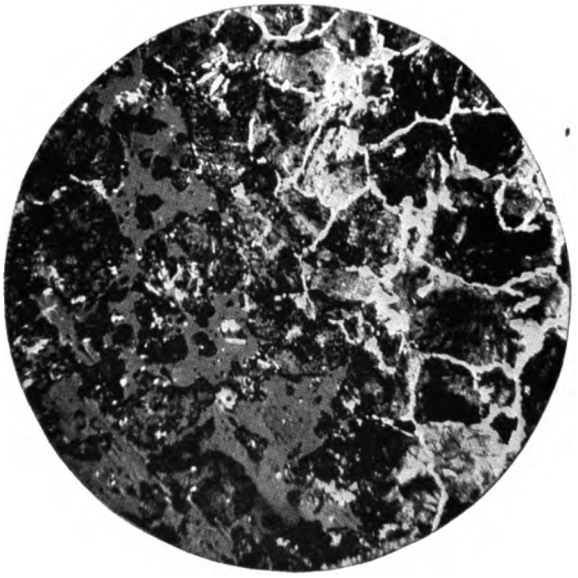

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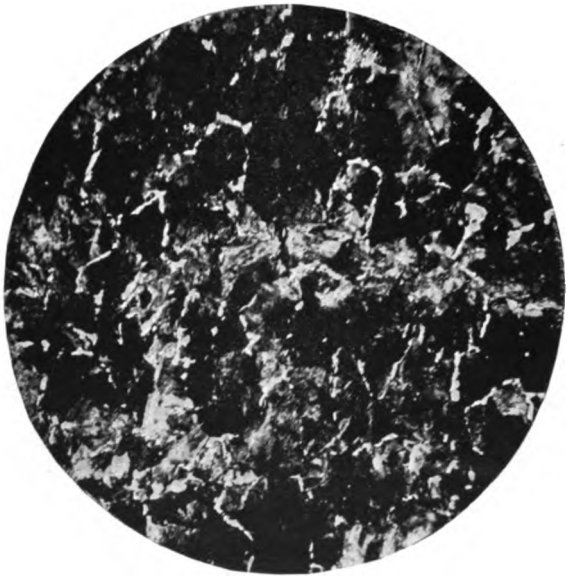

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FIG. 9.—*Rail M5–F from comparison ingot (continued)*

Typical microstructure of the metal of rail M5–F at positions indicated in the sulphur print. × 50, reduced from × 100

(*a*) Structure at the edge of a pronounced pipe, consisting of eutectoid steel contaminated with slag and bordered by a layer of hypoeutectoid metal

(*b*) Moderately finely grained metal, consisting of pearlite with a trace of ferrite, and containing some slag intrusions

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FIG. 10.—*Rail M4-F from comparison ingot*

Sulphur print, showing soft streak in web at F position (bottom of ingot). × 1

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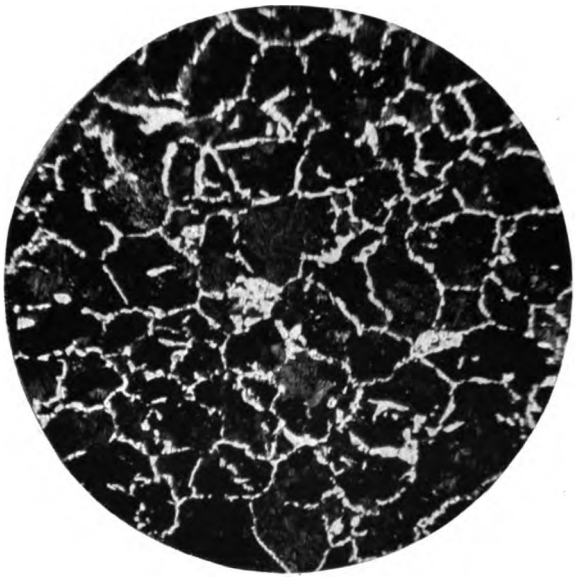

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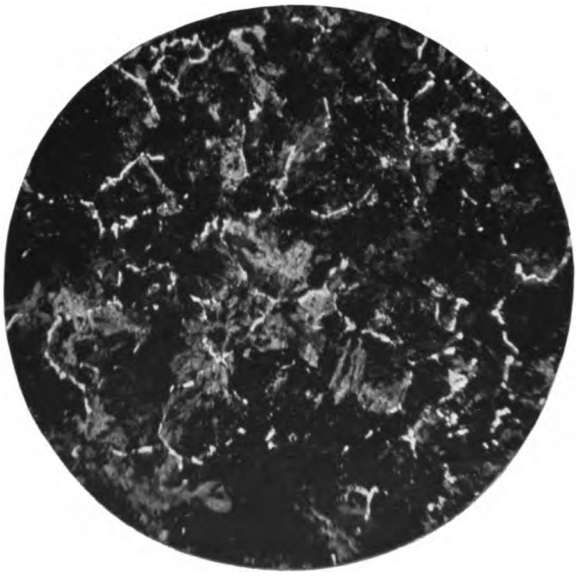

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FIG. 10.—*Rail M4–F from comparison ingot (continued)*

Typical microstructure of the metal of rail M4–F at the positions indicated in the sulphur print. × 50, reduced from × 100
(*a*) Streak of soft metal in the web, consisting of pearlite crystals surrounded by well developed ferrite envelopes
(*b*) Moderately finely grained metal consisting of pearlite and a trace of ferrite

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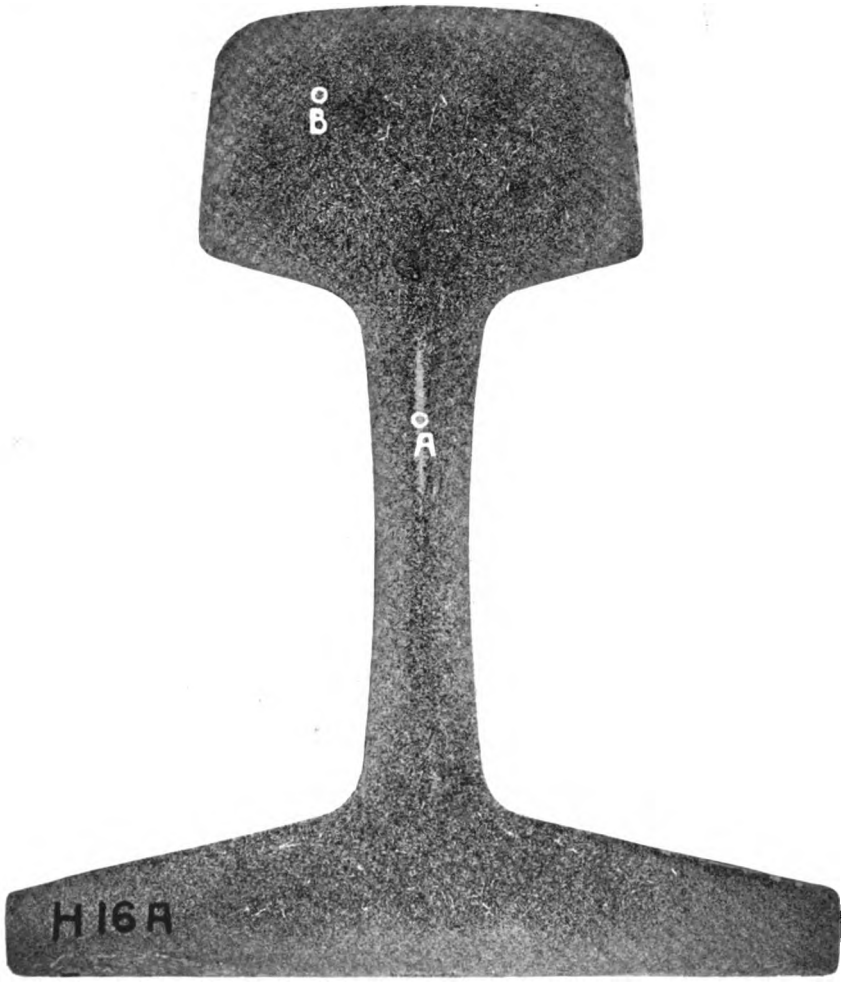

FIG. 11.—*Rail H16–A from sink-head ingot*

Sulphur print, showing slight segregation in upper part of ingot (A position). × 1

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a

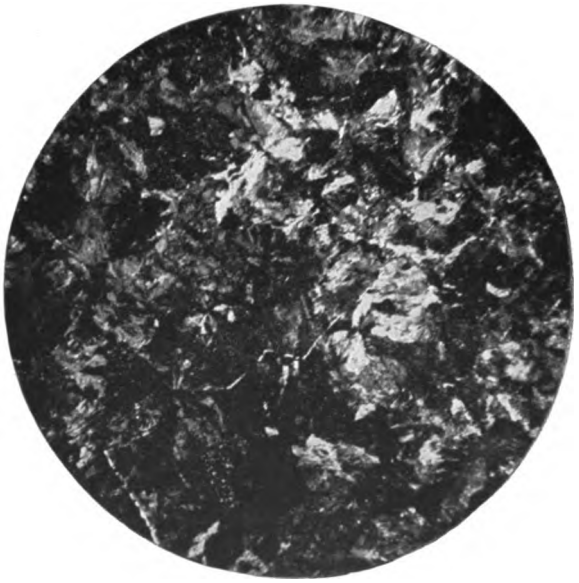

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FIG. 11.—*Rail H16–A from sink-head ingot (continued)*

Typical microstructure of the metal of rail H16-A at positions indicated in the sulphur print. × 50, reduced from × 100
(*a*) Eutectoid metal comprising a slightly segregated streak in the web of the rail
(*b*) Average finely grained structure, pearlite with a trace of ferrite

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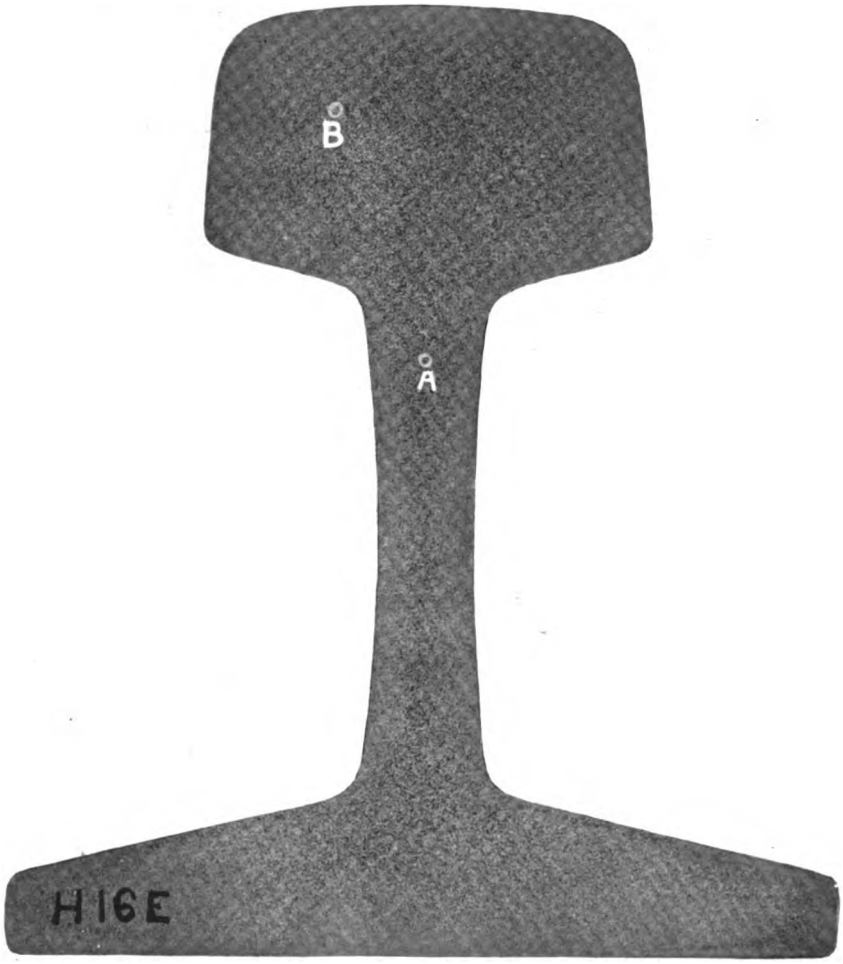

FIG. 12.—*Rail H16–E from sink-head ingot*

Sulphur print, showing sound steel in rail from bottom of ingot (E position). × 1. The rail is homogeneous

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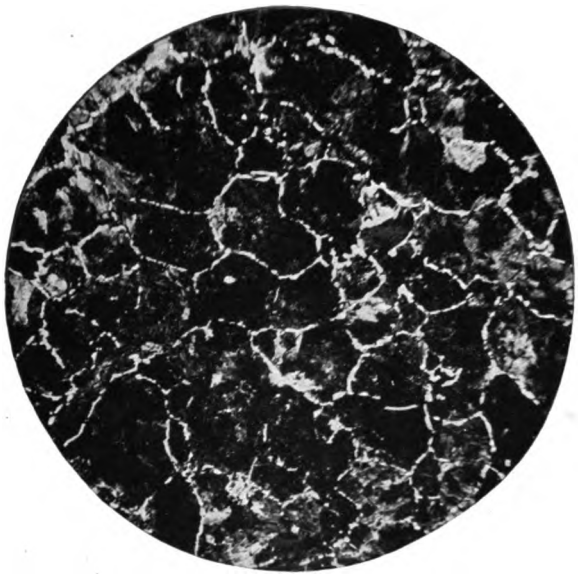

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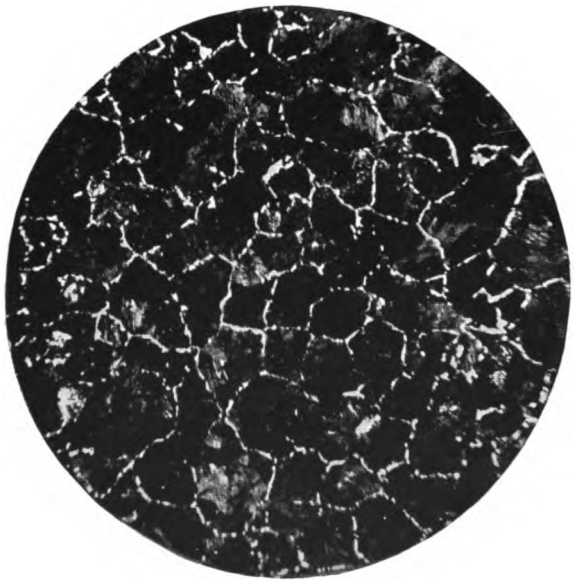

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FIG. 12.—*Rail H16–E from sink-head ingot (continued)*

Typical microstructure of the metal of rail H16-E at the positions indicated in the sulphur print. × 50, reduced from × 100
(*a*) Moderately finely grained metal showing a thick ferrite network
(*b*) Finely grained metal, pearlite with a ferrite network

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in less segregation, greater homogeneity as shown by sulphur prints and etching, less amount and variability in discard to sound steel, suggests that the question of the significance and interpretation of the drop test may well be raised with propriety. The greater ductility and more uniform behavior under the drop test of the steel from the comparison ingots would appear to be due more to the chemical composition, and especially to the nickel and chromium content of this Mayari steel, than to any factor traceable to manufacture, which in the case of the comparison ingots is seen to divide them sharply into three groups of very distinct physical characteristics—each a heat—which fact, the drop test is unable to reveal.

8. METALLOGRAPHIC TESTS

The metallographic tests throw additional light on the question of segregation and soundness, and especially with reference to the presence of slag inclusions, streaks, and seams. They also furnish evidence as to the uniformity of the operations in the rolling mill, although there appears also to be a distribution of structure for each ingot characteristic of its manufacturing processes preceding the rolling.

The smoothness and evenness of tone of the sulphur prints of all rail sections from the Hadfield sink-head ingots, which are characteristic of the lot (Figs. 11–12), even from sections in the drop-test portion near the top of the ingot, is in striking contrast to the markedly irregular sulphur prints from comparison rails (Fig. 8), except for a few below the center of some of these ingots. Most of the rails from the Maryland ingots show very pronounced irregularities in terms of the sulphur prints, some of them from the bottom of ingot showing well-defined pipes (Figs. 9 and 10).

As to the microstructure of the steel, here again the sink-head ingots show much the greater uniformity. This is especially true for the upper portions of the ingot, where differences would be expected.

The parallelism of segregation and type of structure is, in general, very close. Thus in the extreme top portion of the ingot there appears a fine-grained, hypereutectoid structure in the segregated area. The main differences in structure across the section, except for these extreme cases accompanying marked segregation, are usually limited to differences in grain size, pearlite with a ferrite network, the smaller-sized grain accom-

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panying the higher carbon. This is the predominant characteristic of the microstructure of the rails from the sink-head ingots.

The rails from the comparison ingots have, below the segregated area, about the same structural characteristics as those from the sink-head type, except that for many of them the ferrite network is less pronounced and in some cases almost absent. The rolling and finishing temperatures of the ingots were not widely different and do not appear to play any considerable rôle in determining differences of structure. The manganese content, usually somewhat higher in the Hadfield ingots, apparently tends to accentuate the sharp, definite boundaries of ferrite surrounding the pearlite grains. A number of typical photomicrographs are shown in Figs. 8–12, inclusive.

9. EXAMINATION OF SPLIT INGOTS AND BLOOMS

(*a*) SINK-HEAD INGOT H37.—Sink-head ingot H37 was cut in halves at Sheffield and Sir Robert Hadfield reports:

> The whole of this ingot could be worked into rails after cutting off 10 per cent discard. The ingot is perfectly sound and represents the average quality of the whole of the 36 ingots sent to be rolled into rails.
>
> The capacity of the sink-head cavity of this ingot was 8125 cubic centimeters, and the percentage of settling, by weight in pounds—that is, of the total weight of the ingot—was 2.58 per cent. The weight of the material which passed from the head portion into the ingot itself was about 130 pounds. This ingot was selected because it showed the lowest cavity percentage; namely, 2.58 per cent.

(*b*) SINK-HEAD INGOT H32.—Sink-head ingot H32 was cut in halves at the Bureau of Standards. A photograph of a sulphur print of a longitudinal half section of the ingot is shown in Fig. 7. In Figs. 18–21, inclusive, are shown the location of drill holes for chemical analyses of this ingot, as well as the results of the analyses. Only carbon, phosphorus, silicon, and sulphur surveys were made, as the rail analyses indicated that there was no appreciable segregation of manganese in similar ingots. The results obtained indicate that with about 9 per cent discard, steel free from piping and appreciable segregation is obtained. Carbon showed a high degree of segregation (0.68 to 1.13 per cent), but as will be noted, this is confined to the extreme upper portion of the ingot. The high carbon content at this point is probably due to the fact that the steel in the top part of the ingot absorbed carbon from the charcoal used, together with air blast, to keep the ingot hot on its upper surface while cooling. Phosphorus, silicon, sulphur, and manganese are remarkably uniform in this sink-head type of ingot.

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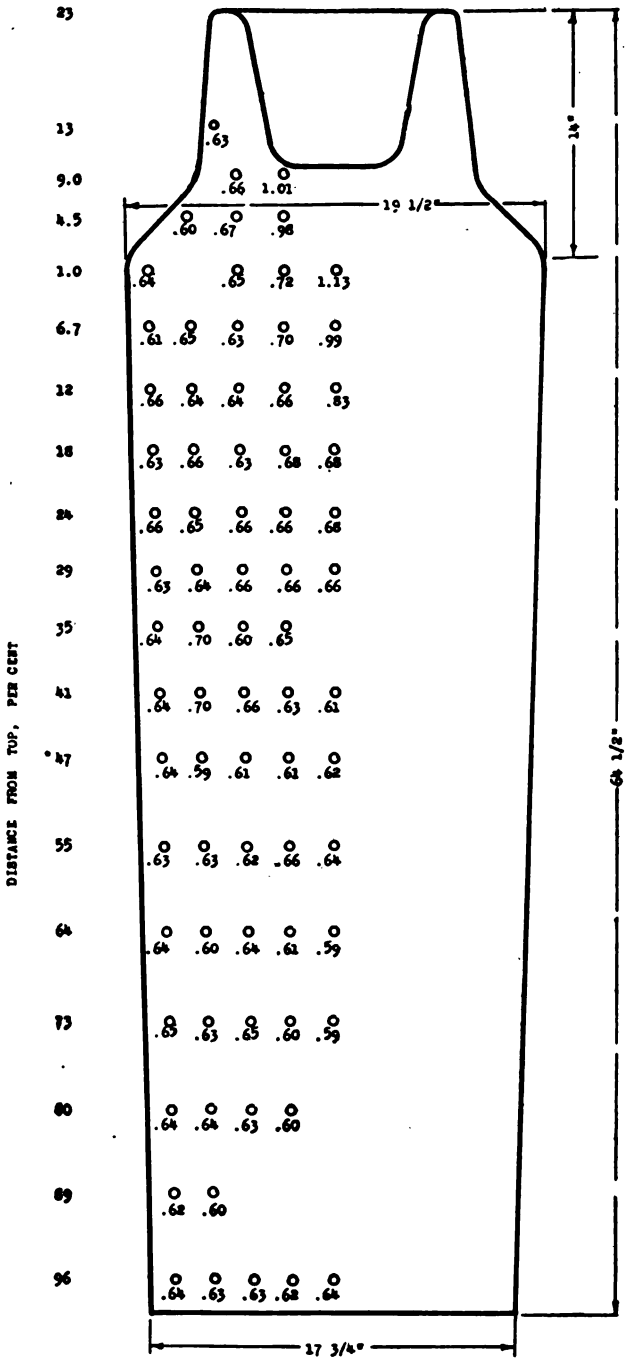

FIG. 18.—*Section of Hadfield split ingot No. 32, showing percentage of carbon*

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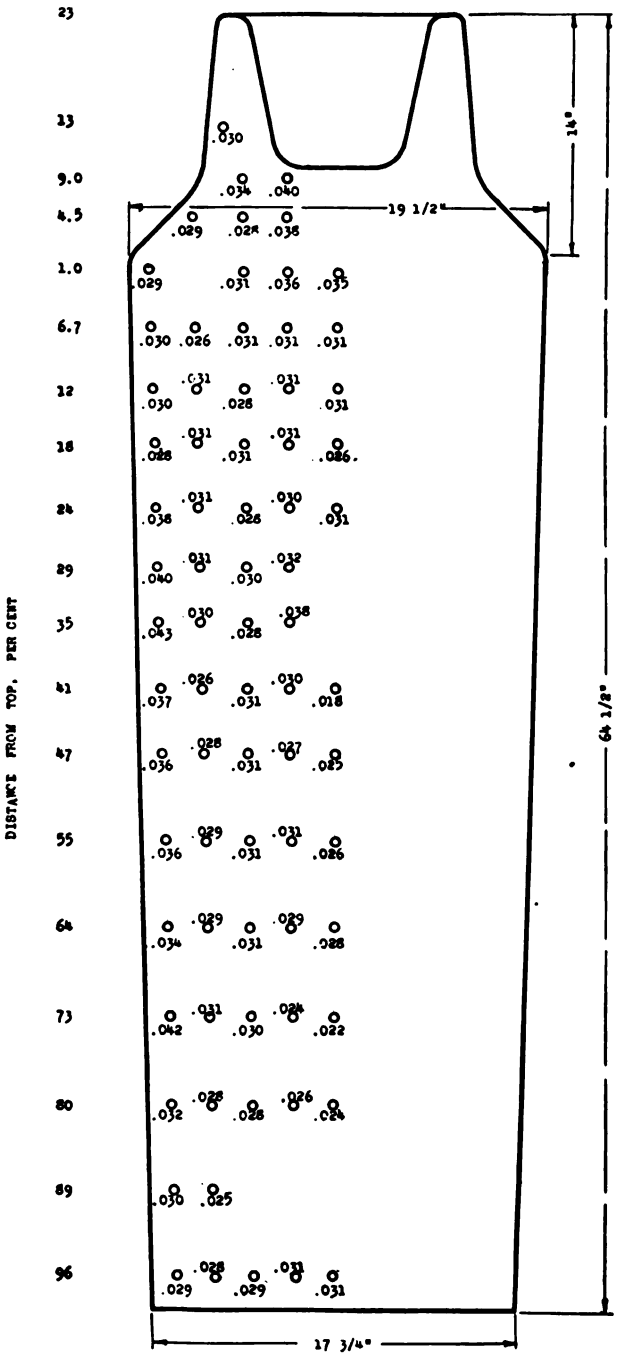

FIG. 19.—*Section of Hadfield split ingot No. 32, showing percentage of phosphorus*

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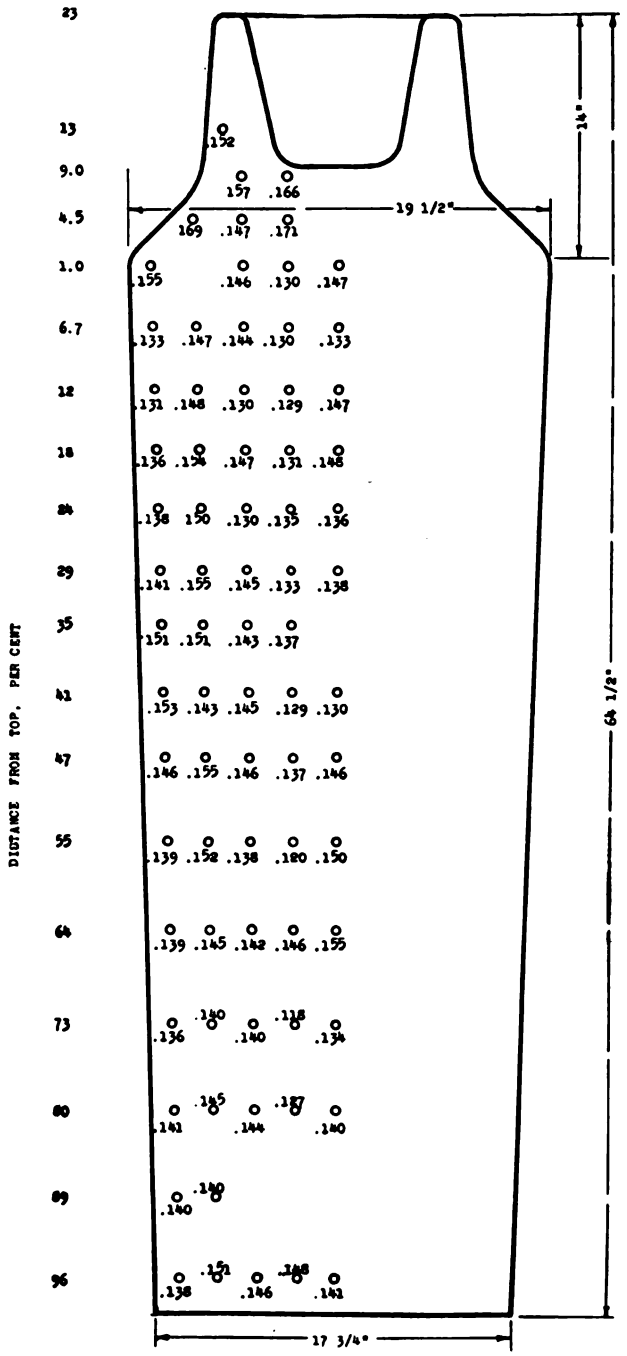

FIG. 20.—*Section of Hadfield split ingot No. 32, showing percentage of silicon*

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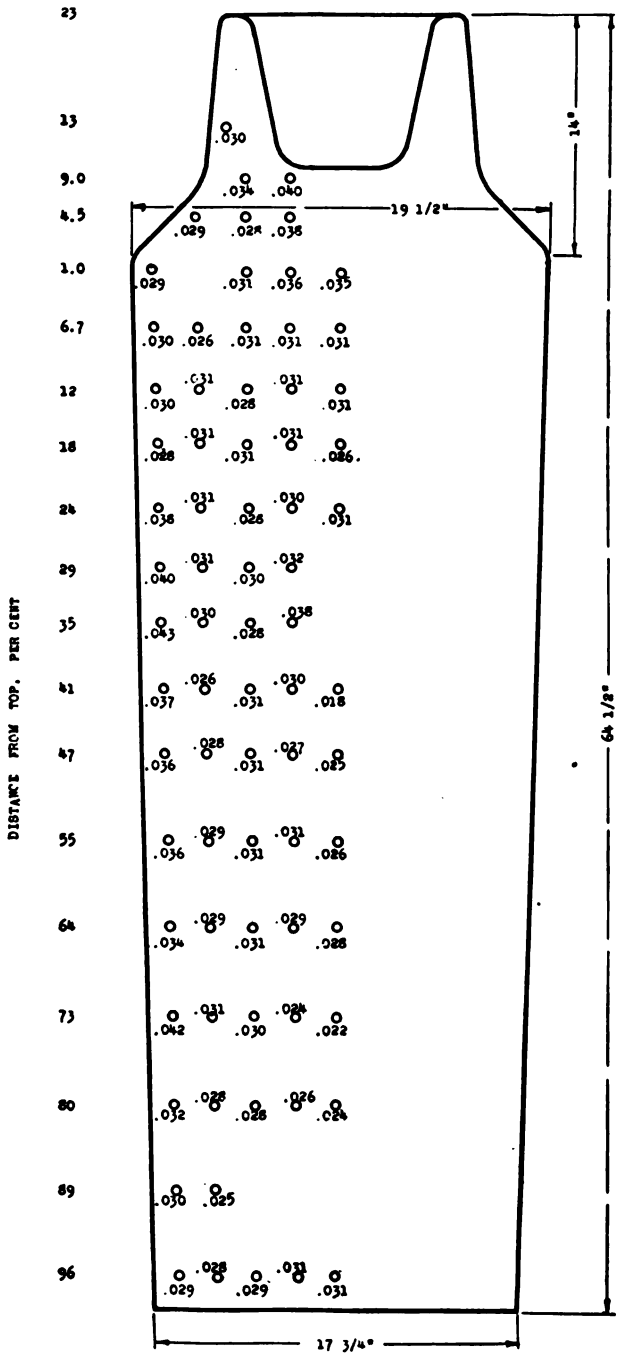

FIG. 19.—*Section of Hadfield split ingot No. 32, showing percentage of phosphorus*

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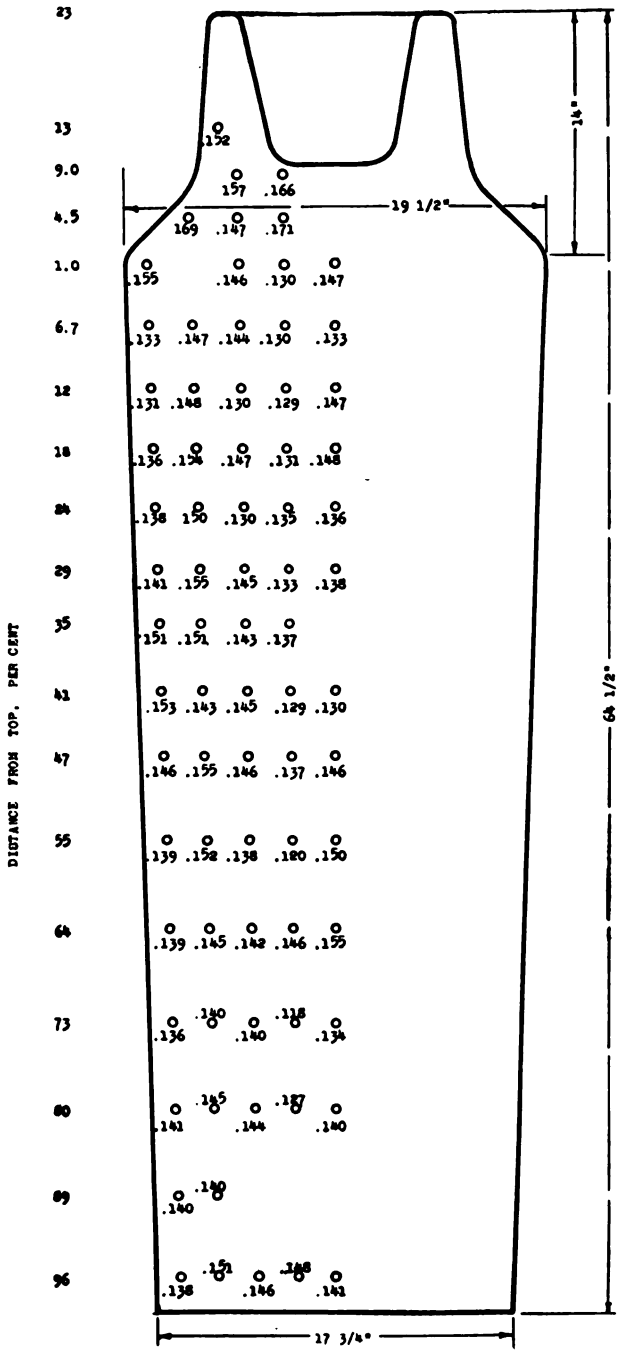

FIG. 20.—*Section of Hadfield split ingot No. 32, showing percentage of silicon*

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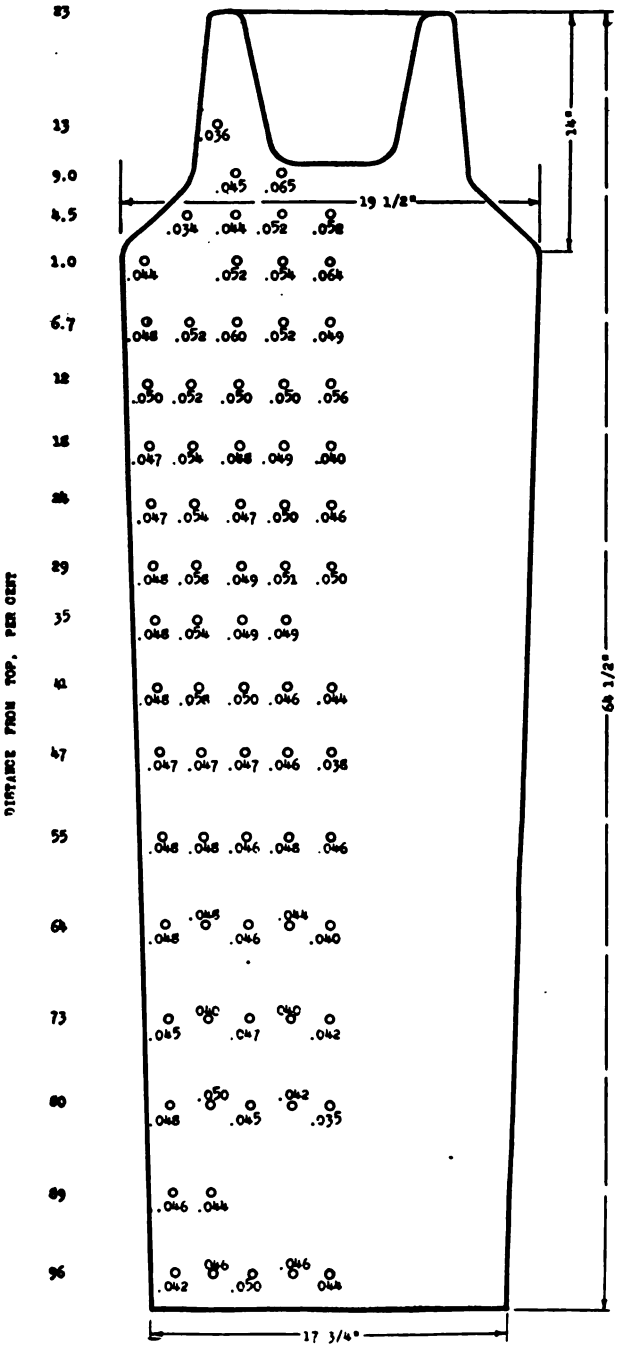

FIG. 21.—*Section of Hadfield split ingot No. 32, showing percentage of sulphur*

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(*c*) SINK-HEAD INGOT H5 AND COMPARISON M10 TOP BLOOMS.—The top blooms from ingots H5 and M10 were split longitudinally at the Bureau of Standards and one half of each used for sulphur prints (Figs. 5 and 6) and the other half of each for a chemical survey (Figs. 22 and 23) of carbon, phosphorus, silicon, and sulphur segregation.

As in the case of the sulphur prints from the rails, the sulphur print from the bloom of the sink-head ingot is of even tone, while the comparison bloom shows decided irregularities (Figs. 5 and 6). In addition, the latter bloom has an inclosed pipe some 12 per cent below the top.

The chemical survey shows the Hadfield bloom to be very uniform. The maximum carbon is 0.90 per cent, and this is only 4 per cent from the top of the ingot. The comparison bloom, according to chemical analyses, is also quite uniform except for the sulphur content, which is very irregular. This element attains a maximum of 0.166 per cent at a distance of 12 per cent from the top, while it is as high as 0.12 per cent at a distance of 21 per cent.

10. SPECIAL NICKEL CHROMIUM SINK-HEAD INGOT

A special sink-head ingot, of the same dimensions and type as the other sink-head ingots but containing chromium and nickel was rolled into rails so as to permit more accurate comparison with the ingots cast in the ordinary manner. The latter naturally contain small amounts of nickel and chromium from the Mayari ore from which the product was made.

The mechanical and chemical properties of the five rails from this special ingot (H38) are given in Table 23. It will be observed that the yield point (70 791 pounds per square inch), the ultimate tensile strength (143 025 pounds per square inch), and the Brinell hardness (292) are greater, while the elongation (11.3 per cent) and reduction of area (19.06 per cent) are less than those of the average sink-head ingot. Under the drop test the rails from this ingot behave similarly to the comparison ingots.

From the standpoint of composition the steel from this ingot differs from that of the average sink-head ingot in that it is of higher carbon (0.73 per cent) and lower phosphorus (0.016 per cent) and sulphur (0.018 per cent); also in that it contains small amounts of nickel (0.19 per cent) and chromium (0.11 per cent).

It was necessary to discard 17.6 per cent of this ingot because of pipe, as against 9.1 per cent for the average sink-head ingot.

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TABLE 23.—Mechanical and Chemical Properties of the Special Sink-Head (Nickel and Chromium) Ingot

Rail	Tension tests				Hardness	
	Yield point	Ultimate strength	Elongation, per cent in 2 inches	Reduction of area	Brinell	Scleroscope
	Lbs./in.²	Lbs./in.²		Per cent		
H38-A	70 030	142 725	12	18.79	274	33
H38-B	70 700	143 400	11	18.59	298	32
H38-C	70 950	143 450	12	18.54	318	29
H38-D	71 445	143 300	11.5	18.38	283	33
H38-E	70 830	142 250	10	21.00	287	31
Average	70 791	143 025	11.3	19.06	292	32

Rail	Results of chemical analysis at "O"; per cent								
	Carbon			Mn	P	Si	S	Ni	Cr
	O	M	Seg.						
H38-A	0.741	0.741	0	0.91	0.024	0.179	0.018	0.19	0.10
H38-B	.731	.729	−0.27	.95	.027	.177	.019	.19	.10
H38-C	.728	.735	+.96	.93	.026	.179	.018	.19	.12
H38-D	.734	.726	−1.09	.94	.027	.185	.018	.18	.10
H38-E	.731	.730	− .14	.96	.025	.180	.019	.19	.11
Average	.733	.732	− .11	.94	.026	.180	.018	.19	.11

IV. SERVICE OF RAILS FROM THIS INVESTIGATION

The Pennsylvania Railroad, through A. W. Gibbs, reports as follows on the service of the rails from the earlier investigation [6] and from the present one:

In January, 1915, the Maryland Steel Co. shipped free for test 5 No. 1 33-foot P. S. 100-pound open-hearth steel rails rolled from ingots furnished by Sir Robert Hadfield to the Bureau of Standards. These rails were laid in the Eastward freight track on Horseshoe Curve, January 20, 1915, and removed May 15, 1915. The average abrasion of the Hadfield rails was 1.16 square inches and that of the ordinary rail 0.78 square inch. None of these rails failed.

In September, 1915, the Maryland Steel Co. rolled 46 No. 1 and 61 No. 2 P. S. 100-pound rails (0.33 track mile) from imported Hadfield sink-head ingots and at the same time rolled 53 No. 1 and 4 No. 2 P. S. 100-pound rails (0.18 track mile) from its own ingots for comparison. The 46 No. 1 Hadfield and 53 No. 1 Maryland special rails were laid November 26, 1915, in the Eastward passenger and freight track on the 5° 30′ curves between Spruce Creek and Union Furnace, Middle Division, and removed October 23, 1916. The 61 No. 2 Hadfield and 4 No. 2 Maryland special rails were laid on a 2° curve in the Eastward Low Grade track, Philadelphia Division, west of M. P. 69, and are still in track. The average abrasion of the No. 1 Hadfield rails was 0.42 square inch; that of the Maryland special rails 0.37. The rail on Low Grade was still in track October 31, 1919; abrasion was then 16.1 per cent for Hadfield and 9.7 per cent for Maryland special.

There have been no failures of Hadfield rails or of the Maryland specials.

There is evidently no economy in the Hadfield process as regards wear, and it would require a much more extended trial to determine the relative freedom from failure.

[6] See footnote 1.

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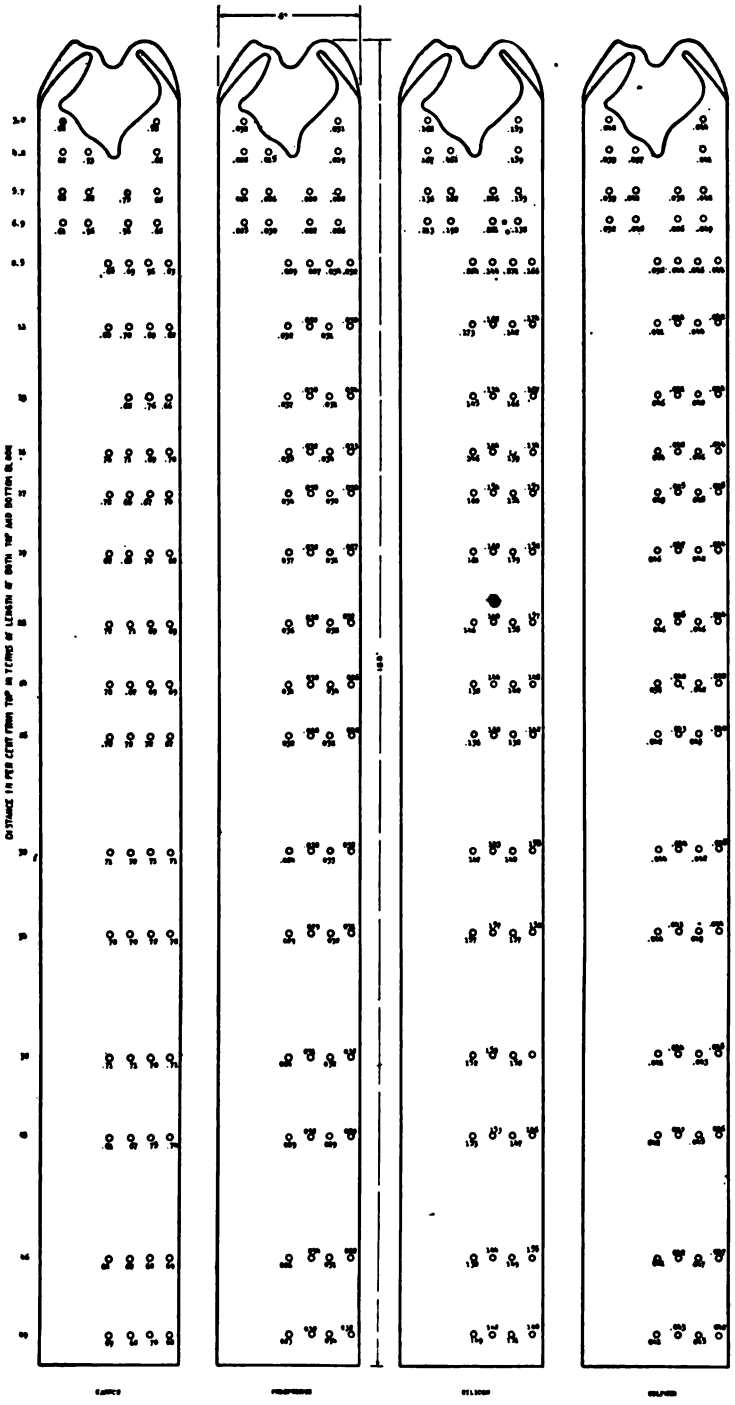

FIG. 22.—*Sections of top bloom from Hadfield ingot H5, showing chemical analysis of various positions*

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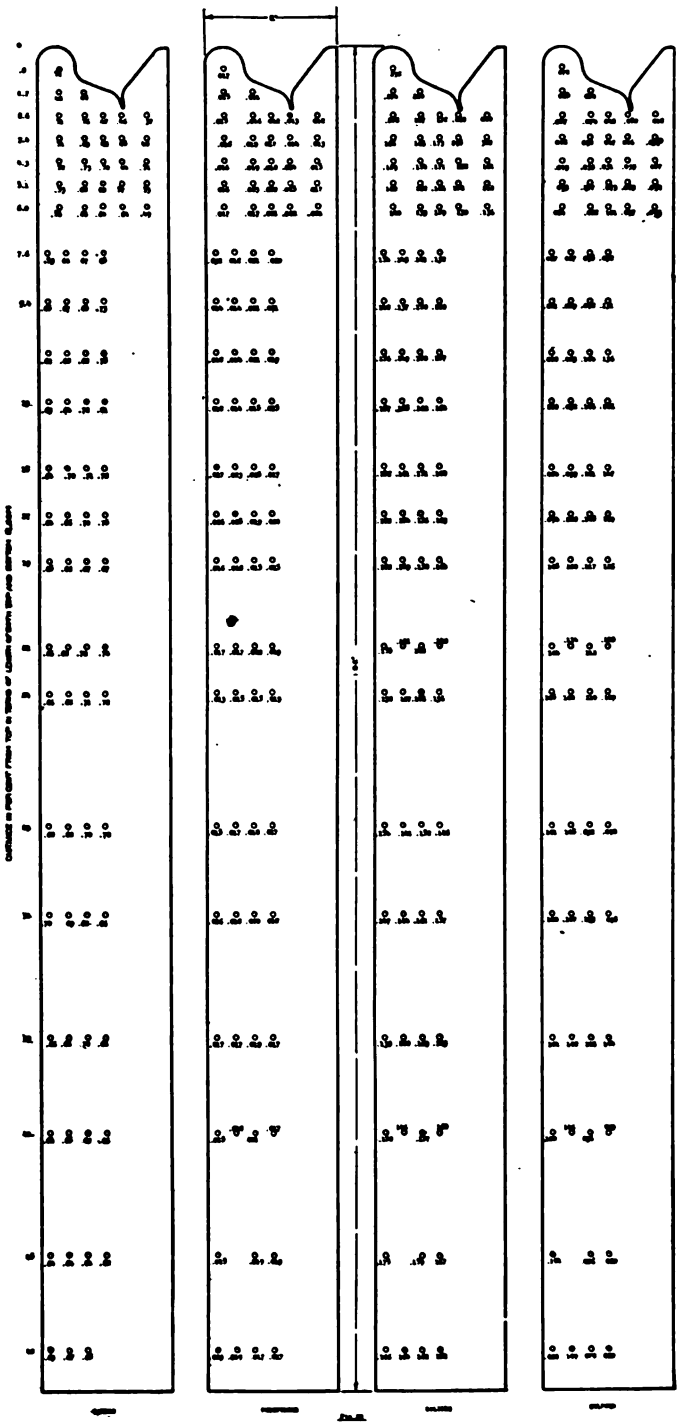

FIG. 23.—*Sections of top bloom from Maryland ingot M10, showing chemical analysis of various positions*

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The Maryland rails may owe their less abrasion to their content of nickel and chromium. It would appear to be unfair to draw any general conclusions as to performance in service from so few rails as this investigation furnished, although it would not be expected that the Hadfield type of sink-head ingots would furnish any rails showing structural defects.

V. SUMMARY AND CONCLUSIONS

The object of this investigation was to determine the relation of ingot practice to the properties of rails from such ingots, and in particular the amount of total discard necessary to obtain rails free from piping and segregation above 12 per cent which had been rolled from steel made in accordance with varying melting, casting, and ingot practices.

To that end, 35 ingots made by the converter process at Hadfield's, Sheffield, England, and cast by the sink-head process with large end uppermost were shipped to Sparrows Point, Md., and rolled into rails in comparison with 15 rail ingots made in the ordinary manner with the small end uppermost. Each sink-head ingot, of 5300 pounds weight, and deoxidized with aluminum in the mold, represented a separate heat of converter steel, and all the heats and ingots were made in the same manner. The composition and properties of these ingots were of remarkable uniformity. The comparison ingots, of 7300 pounds each, were from 3 separate open-hearth heats, an intentional variation being made in the open-hearth and casting practice for each. Five ingots were selected from each of these 3 heats. Thus in reality, comparison was made of 4 different kinds of steel, of very nearly the same composition and physical properties and of two types of ingot form.

The comparison was made by rolling most of the ingots into rails and taking test specimens at each rail cut, as well as from a considerable portion of the upper part, in 5-foot steps, of the rail bar from each ingot. In this way there was obtained a detailed physical, chemical, and metallographic survey of each ingot, and it was possible to exactly delimit the regions of sound and homogeneous from those of unsound and segregated steel. Two complete sink-head ingots were cut longitudinally and examined, as well as representative blooms from both sink-head and ordinary ingots.

The results obtained indicate a decided superiority of the sink-head ingots over the comparison ingots as made of three grades of steel (Tables 17 and 18), although the sink-head ingots suffered from the disadvantage of having gone cold before rolling. The

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Hadfield type of ingot required a total discard of only 18.4 per cent on the average (13 per cent top discard to eliminate piping and segregation above 12 per cent) while the average ingot of the ordinary type for rails required a total discard of 43.9 per cent (26 per cent top discard), with great variations dependent upon the furnace and ingot practices.

The comparison ingots from heat M1–M5 of nondeoxidized rising steel, chilled on top of ingot by cast-iron caps, required excessive discard to eliminate positive segregation at the top and negative segregation at the bottom of the ingot, the latter often accompanied by dangerous inclosed pipes.

The second heat (M6–M10) made of rising steel deoxidized with aluminum in the molds; the ingot tops of which were cooled with water, required the least total discard of the three heats. It was more subject to piping and less to segregation than the first heat of ingots made in the usual manner.

The third heat (M11–M15) made of quiet or "killed" steel, was not chilled on top with water or caps, and was deoxidized with aluminum in the molds. The ingots of this heat required an intermediate amount of total discard when compared to the first and second heats; this heat was the only one for which a greater top discard was required to eliminate piping than to eliminate segregation above 12 per cent. One of the ingots of this third heat contained a small pipe at the bottom, and all the rails from the middle and bottom of the ingots showed high negative segregation.

The variation of physical properties throughout the length of each ingot is characteristic not only of the type of the ingot, as sink-head or ordinary, but also of the state of the steel when cast and of the ingot practice.

It has been established in the foregoing that, after removal of the top discard of 13 per cent, the Hadfield type of sink-head ingot is free from piping and undue segregation. The ordinary type of ingot, cast small end up without sink head, as is usual for rail ingots, requires an average top discard of 26 per cent, and the remainder of the ingot is liable to contain inclosed piping and excessive segregation. Defective rails, from the middle and bottom portion of the ingot, are not detected with certainty by means of the test for existing rail specifications, and as a result of this uncertainty rails containing pipes or excessive segregation may get into service with disastrous results.

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The surface condition of the rails from the sink-head ingots was not as good as that of the ordinary ingots, but this is not considered an essential characteristic of rails from such ingots.

The markedly differing characteristics of the three heats of comparison ingots lead one to raise the question whether or not it might be advisable to specify, at least in some degree, the methods of steel manufacture or of ingot practice for rails and similar products on which the safety of the traveling public depends

While it is not claimed that the use of the sink-head process for the manufacture of ingots will solve all rail problems, it is maintained that its adoption would be a step in the right direction in view of the present heavy casualties and property losses on American railroads. The necessary changes in mill operations, it is believed, could be made without too great difficulties.

The service results, showing no failures to date, in the few rails of this investigation can not be considered conclusive one way or another.

It would be of the greatest practical importance in tracing statistically the origin of rail failures, if, in the reports to the rail committee of the American Railroad Engineering Association, they be grouped with reference to their position in the ingot, and in addition the steel manufacturing process be reported, at least briefly by such notations as rising steel capped with plates; quiet steel deoxidized with x ounces of aluminum per ton, etc. If then it be found, for a given kind of steel, there is a greater or less tendency for certain types of failure to predominate or to exist in certain definite regions in the ingot, corrections could then be made intelligently in the manufacturing process. The results of the present investigation indicates very strongly that one should expect the four types of steel and two types of ingot examined, in view of their sharply differing characteristics, to behave differently in service.

WASHINGTON, June 7, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

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No. 179

ELECTRIC-ARC WELDING OF STEEL:
I. PROPERTIES OF THE ARC-FUSED METAL

BY

HENRY S. RAWDON, Physicist
EDWARD C. GROESBECK, Physical Chemist
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Bureau of Standards

NOVEMBER 15, 1920

PRICE, 15 CENTS

Sold only by the Superintendent of Documents, Government Printing Office
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By Henry S. Rawdon, Edward C. Groesbeck, and Louis Jordan

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I. INTRODUCTION

The daily advances in the variety and number of applications of electric welding bespeak the continually increasing industrial importance of this method of joining and repairing metals. Of the two forms of electric welding in common industrial use, the arc-fusion method is of far more general application than is electrical-resistance welding; arc welding, as is evident from its nature, can be applied in a multitude of ways to as many different welding projects, while the second method, which depends upon the contact resistance of the two parts which are to be joined together, finds its greatest applications in "repeat" welding.

Resistance welding is closely related to the familiar weld of the smith's forge in that the two surfaces to be joined are heated to the welding temperature, the electric current being the source of the necessary heat, and then pressed into intimate contact while

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hot. The principle of arc welding, however, is very different; the two parts are joined together by fusion, the fused metal being supplied from an outside source rather than from the parts which are to be welded. Often the addition of a very considerable amount of metal is necessary, which metal is supplied by the fusion accomplished by means of the heat of the electric arc. A layer of considerable thickness often exists between the two parts joined together.

II. AIM OF THE INVESTIGATION

The application of arc welding in the shipbuilding industry is one of the most important and extensive applications which has as yet been proposed for this method. During the year 1918, at the request of and with the cooperation of the Welding Research Subcommittee of the Emergency Fleet Corporation, an extensive program was outlined by the Bureau of Standards for the study of this type of welding. The principal object of the research was to determine in an empirical way by actual welding tests carried out by skilled operators the relative values and merits of the different types of electrodes or welding pencils which are available commercially. Due to changed conditions, however, at the close of the year 1918 the original program was modified and shortened very considerably. The results of the investigation here given relate principally to the nature and characteristic properties of the weld, and in particular those of the 'fused-in" metal.

In drawing up the modified program it was decided to make the study of the characteristic properties of the "fused-in" metal the primary object of the investigation, the study of the merits of the different types of electrodes being a secondary one. Since the metal of any weld produced by the electric-arc fusion method is essentially a casting because there is no refinement possible as in some of the other methods, it is apparent that the efficiency of the weld is dependent upon the properties of this arc-fused metal. Hence a knowledge of its properties is of fundamental importance in the study of electric-arc welds.

Though the results obtained throughout the entire series of examinations are not so conclusive as might be desired, it is believed that they contribute very materially to our knowledge of the properties of the "fused-in" metal, which conditions the serviceability of the electric-arc weld.

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III. PRELIMINARY EXAMINATIONS OF ELECTRIC-ARC WELDS [1]

Numerous articles have appeared in the technical literature bearing on the subject of electric-arc welding. Most of these, however, are devoted to the technique and comparative merits of the method, manipulations, equipment, etc., rather than to the study of the characteristics of the metal of the weld itself. The information on this phase of the subject is rather meager.[2]

A considerable number of examinations was made of welds prepared by means of the electric-arc process and representative of different conditions of welding. Most of these were of a general miscellaneous nature, and the results do not warrant including a description of the different specimens here. One series of particular interest, however, may well be referred to in detail. As part of this study the Welding Research Subcommittee submitted to the Bureau of Standards for examination a number of welds of ship plate representative of English practice, some of the welds being considered as very superior examples of welding, others being of a decidedly inferior grade. In Tables 1 and 2 are given the results obtained by the mechanical tests made upon these specimens. The welding was done by skilled operators by means of special brands of electrodes (welding pencils), the trade names of which, however, have been omitted from the tables. In addition to the mechanical tests the specimens were examined very carefully microscopically. The results are not included, however, as the structural features of the material did not differ from those discussed later in Section VII. The results of the mechanical tests given are of value in that they are indicative of the average mechanical properties which should be expected in electric-arc welds of satisfactory grade and of the shape and size of those examined.

---

[1] These examinations, as well as the preparation of many of the specimens used later in the investigation, were made by B. Hurvitz, formerly assistant chemist of the Bureau of Standards.

[2] A fairly complete bibliography on the mechanical properties and structure of the metal of the weld and also on nitrogen in iron and steel has been given in the appendix. The necessary references throughout the text are made by quoting the bibliography title number.

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TABLE 1.—Mechanical Properties of 12 Good Welds [a]

No.	Average voltage, direct current	Average current	Type of weld [b]	Thickness of plate	Ultimate tensile strength	Elongation	Gage length	Fracture
		Amperes		Inch	Lbs./in.²	Per cent	Inches	
1	60	120	Vertical...	3/4	51 450	7	8	In weld, fine crystalline, some holes
2	60	120	...do.....	3/4	53 200	11	8	Do.
3	[c] 75	110	Flat.....	5/8	57 430	12	8	Outside weld, few holes.
4	[c] 75	110	...do.....	5/8	54 210	14	8	Do.
5	110	70	...do.....	1/4	60 610	5.5	6	Do.
6	110	125	...do.....	3/4	59 000	6	6	In weld, crystalline, fine to coarse, few holes
7	110	150	...do.....	1	52 570	6	6	In weld, very fine crystalline, few holes
8	110	100	Overhead.	1/2	59 470	4	6	In weld, fine crystalline, many holes
9	110	120	Flat.....	1/2	55 460	6	6	In weld, crystalline, fine to coarse, few holes
10	110	120	Vertical...	1/2	49 030	3.7	6	In weld, coarse crystalline, few holes
					Cold-bend test [d]			
					Load at yield	Load at break	Angle of bend	
					Pounds	Pounds	Degrees	
11	110	70	Flat.....	1/4	850	1 010	105	
12	110	150	...do.....	1	13 530	17 450	20	In weld, fine crystalline, few holes

[a] All the welds were made in steel plate of the thickness shown. Electrodes of the covered type were used. The welds were of 60° V type except the overhead welds, in which a 90° V was used.

[b] Refers to the position of the plates which were being welded together.

[c] Alternating current.

[d] The bend tests were made with the apex of V in tension with a 7-inch span over a pin of 2-inch radius except the ¼-inch plate, for which a pin of 1-inch radius was used.

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TABLE 2.—Mechanical Properties of 12 Inferior Welds [a]

No.	Average voltage, direct current	Average current	Type of weld [b]	Thickness of plate	Ultimate tensile strength	Elongation	Gage length	Fracture
		Amperes		Inch	Lbs./in.[2]	Per cent	Inches	
13	110	120	Flat.....	5/8	32 460	Nil	.....	In weld, fine crystalline, many holes
14	95	105	Vertical...	1	19 890	Nil	.....	In weld, spongy metal, poor junction with metal of plate
15	95	105	Flat.....	1	31 700	.....	.....	Do.
16	60	120	...do.....	1/2	37 290	2	6	In weld, very fine grained with many holes
17	60	110	Vertical...	1/2	38 820	2	6	In weld, coarse crystalline, many holes
18	[c] 75	110	...do.....	7/8	31 360	Nil	.....	In weld, fine to coarse crystalline, many holes
19	[c] 75	110	...do.....	1	27 090	1.4	8	In weld, coarse crystalline, many holes
20	110	120	...do.....	1/2	39 400	3.5	6	In weld, very fine grained with crystalline areas, many holes
21	110	120	Flat.....	1/2	34 650	Nil	.....	In weld, very fine grained, very many holes
22	110	120	...do.....	3/4	35 120	Nil	.....	In weld, very fine to coarse crystalline, many holes
					Cold-bend test [d]			
					Load at yield	Load at break	Angle of bend	
					Pounds	Pounds	Degrees	
23	95	105	...do.....	1	4 930	4 930	Nil	In weld, very many laps and holes
24	95	105	Vertical...	1	4 120	4 610	1	Do.

[a] See note [a], Table 1. [b] See note [b], Table 1. [c] See note [c], Table 1. [d] See note [d], Table 1.

IV. METHOD AND MATERIALS [3]

The specimens required for the study of the mechanical properties of the arc-fused iron were prepared for the most part at this Bureau, direct current being used in the operation. The apparatus used is shown diagrammatically in Fig. 1. By means of the adjustable water rheostat the current could be regulated from 110 to 300 amperes. By the use of automatic recording instruments the voltage and current were measured and records were taken at intervals during the preparation of a specimen.

[3] Acknowledgement is due A. B. Engle, formerly laboratory assistant, for aid in the preparation of many of the specimens.

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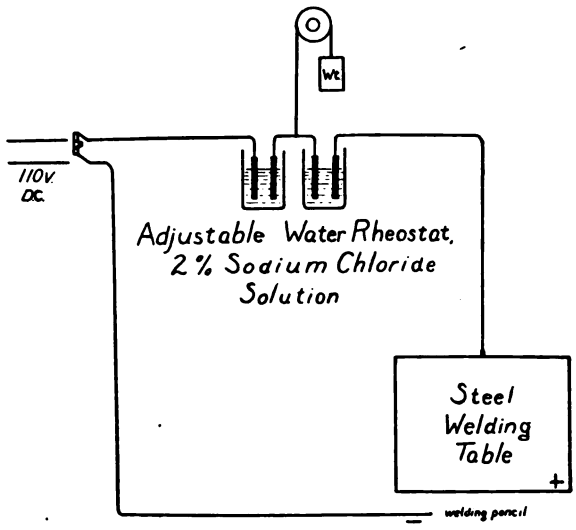

FIG. 1.—*Arrangement of apparatus for welding*

The values of current given below in the tables are those which were desired and were aimed at, the average deviation from the value as recorded by the curves being approximately ±5 amperes.

Since the investigation was concerned primarily with the properties of the arc-fused metal, regular welds were not made. Instead the metal was deposited in a block large enough to permit a tension specimen (0.505 inch diameter, 2-inch gage length) to be machined out of it. Although the opinion is held by some welders that the properties of the metal of an arc weld are affected materially by the adjacent metal by reason of the interpenetration of the two, it was decided that the change of properties of the added metal induced by fusion alone was of fundamental importance and should form the basis of any study of arc welding. The method adopted also permitted the use of larger specimens with much less machining than would have been possible had the metal been deposited in the usual form of a weld.

In the first few specimens prepared (10 in number) the metal was deposited by a series of "beadings" as shown in Fig. 2*a*. The tension specimens cut from the deposited metal were found to be very inferior and entirely unsuitable for the study. This was largely on account of the excessive overheating which occurred, as well as the fact that a relatively "long arc" was

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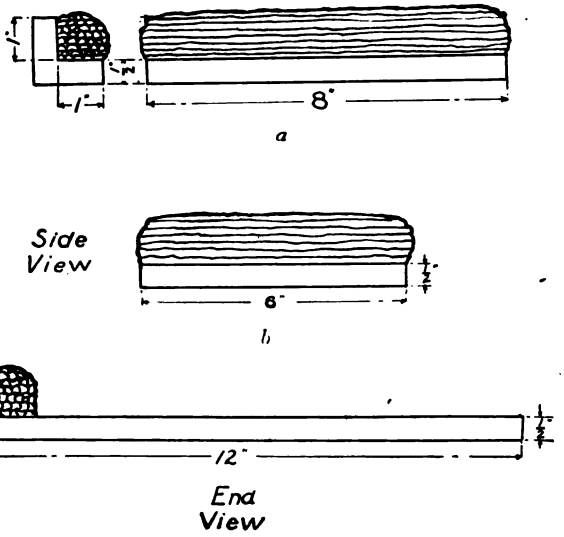

FIG. 2.—*Method of formation of the blocks of arc-fused steel*

a, Early method used, but discarded because of the inferiority of the resulting specimens; *b*, method used in the formation of all specimens, the results of which are given in the following pages

necessary for the fusion in this form. Because of the very evident inferiority of these specimens the results of the mechanical tests made are not given in the tables below (Sec. VI). The method of deposition of the metal was then changed to that shown in Fig. 2*b;* this method also had the advantage in that the amount of necessary machining for shaping the specimens for test was materially reduced. The block of arc-fused metal was built up on the end of a section of one-half inch plate of mild steel (ship plate) as shown. When a block of sufficient size had been formed, it, together with the portion of the steel plate immediately beneath, was sawed off from the remainder of the steel plate. The tension specimen was turned entirely out of the arc-fused metal. No difficulty whatever was experienced in machining the specimens. Fig. 3 shows the general appearance of the block of fused metal as well as the tension specimen turned out of it.

In general, in forming the blocks the fused metal was deposited as a series of "beads" so arranged that they were parallel to the axis of the tension specimen which was cut later from the block. In two cases for purposes of comparison the metal was deposited in "beads" at right angles to the length of the specimen. In all the specimens after the deposition of each layer the surface

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was very carefully and vigorously brushed with a stiff wire brush to remove the layer of oxide and slag which formed during the fusion. There was found to be but little need to use the chisel for removing this layer.

FIG. 3.—*Block of arc-fused metal with tension specimen cut from it. Reduction, approximately ½*

Two types of electrodes were used as material to be fused. These differed considerably in composition (Table 3, Sec. V) and were chosen as representative of a "pure" iron and a low-carbon steel. The two types will be referred to as "A" and "B," respectively, in the tables of results below. They were obtained in the following sizes: One-eighth inch, five-thirty-seconds inch, three-sixteenths inch, and one-fourth inch (A electrodes five-sixteenths inch). It was planned to use the different sizes with the following currents: One-fourth inch—75, 110, and 145 amperes; five-thirty-seconds inch—145, 185, and 225 amperes; three-sixteenths inch—185, 225, and 260 amperes; one-fourth inch (five-sixteenths inch)—300 amperes. The electrodes were used both in the bare condition and after being slightly coated with a deoxidizing and refractory mixture. For coating, a paste of the following composition was used: Fifteen g graphite, 7.5 g magnesium, 4 g aluminum, 65 g magnesium oxide, 60 g calcium oxide. To this mixture was added 120 cm^3 of sodium silicate (40° Be) and 150 cm^3 of water. The electrodes were painted on one side only with

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the paste. The quantity given above was found to be sufficient for coating 500 electrodes. The purpose of the coating was to prevent excessive oxidation of the metal of the electrode during fusion and to form also a thin protective coating of slag upon the fused metal.

Tension specimens only were prepared from the arc-fused metal; the modification and curtailment of the original program rendered this necessary. It is quite generally recognized that the tension test falls very short in completely defining the mechanical properties of any metal; it is believed, however, that the behavior of this material when stressed in tension is so characteristic that its general behavior under other conditions of stress, particularly when subjected to the so-called dynamic tests—that is—vibration and shock, can be safely predicted from the results obtained. In order to supplement the specimens made at this Bureau a series of six was also prepared by one of the large manufacturers of equipment for electric welding to be included in the investigation. These are designated as "C" in the tables below.

V. CHANGES IN COMPOSITION OF ELECTRODES AS A RESULT OF FUSION [4]

In Table 3 below are given the results of the analyses of several typical specimens of the two types of electrodes used for welding and also results of analyses of similar material after arc-fusion.

TABLE 3.—Results of Chemical Analyses of Electrodes Before and After Fusion [a]

Electrode		Carbon		Silicon		Manganese		Phosphorus	
Type	Diameter	Before	After	Before	After	Before	After	Before	After
	Inch	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
A	1/8	0.058	0.046 [b] .031	0.33	0.007 [b] .007	0.042	Tr Tr	0.002	0.005 [b] .005
A	5/32								
A	3/16	.022	.010 [b] .010 .033	.16	.012 [b] .014 .006	.038	Tr [b] Tr 0.069	.002	.003 [b] .002 .012
A	5/16		.050		.11		.014		.011
B	1/8	.15	.027 [b] .024	.06	.008 [b] .100	.47	Tr [b] Tr	.018	.002 [b] .004
B	5/32	.15		.001		.46		.014	
B	3/16								
B	1/4								
C	5/32								

[a] The electrodes which furnished the specimens used for analysis after fusion were not the identical ones used before fusion, but were the same stock.

[b] Results were obtained from the fusion of coated electrodes.

[4] Acknowledgment is due J. R. Cain, chemist, Bureau of Standards, for the method used in the determination of nitrogen. The details of the method, necessary precautions, and apparatus will be described in a separate publication.

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TABLE 3.—Results of Chemical Analyses of Electrodes Before and After Fusion—Continued

Electrode		Sulphur		Copper [a]		Nitrogen [b]	
Type	Diameter	Before	After	Before	After	Before	After
	Inch	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
A	1/8	0.057	0.036 [c] .033			0.0030	0.156 .127 [c] .149 [c] .140
A	5/32					.0035	.140 .123 .124 [c] .121 [c] .119 [c] .113
A	3/16	.040	.033 [c] .035 .043		 0.058	.0040	.126 .131 .133 [c] .127 [c] .131 [c] .134
A	5/16		.026		.063	.0037	.117 [c] .111
B	1/8	.021	 .035 [c] .035			.0032	.152 .132 [c] .141 [c] .135
B	5/32	.017				.0035	.124 .121 .117 [c] .122 [d] .132 [c] .123
B	3/16					.0022 .0025	.119 .111 .112 [c] .106 [c] .108 [c] .094
B	1/4					.0014 .0022	
C	5/32						.133 .098

[a] Determinations for copper were not carried out upon the unfused electrodes.

[b] Each of the results reported in the "after" fusion columns, excepting [c], is the average of two determinations made on one separate specimen.

[c] Results were obtained from the fusion of coated electrodes.

[d] Average of nine determinations.

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It will be noted that the general effect of the fusion is to render the two materials more nearly the same in composition. The loss of carbon and of silicon is very marked in each case where these elements exist in considerable amounts. A similar tendency may be noted for manganese. The coating with which the electrodes were covered appears to have had but little influence, if any, in preventing the oxidation of the carbon and other elements.

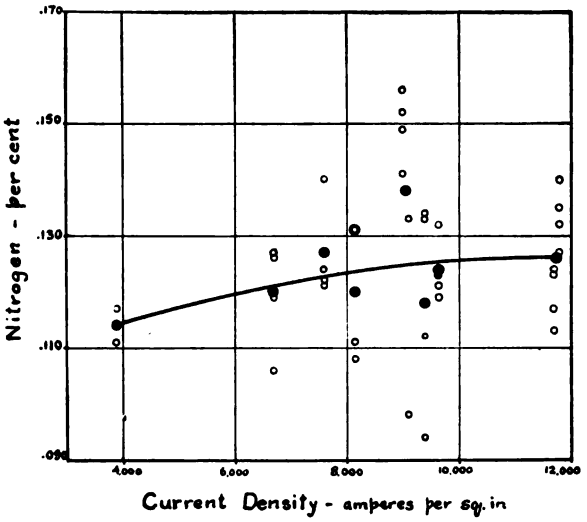

FIG. 4.—*Relation of current density to nitrogen content of the arc-fused steel*

The most noticeable change in composition is the increase in the nitrogen content of the metal. In general the increase was rather uniform for all specimens. In Table 4 are summarized the results of the nitrogen determinations together with the corresponding current density used for the fusion of the metal. In Fig. 4 the average nitrogen content found for the different conditions of fusion is plotted against the corresponding current density. Though no definite conclusion seems to be warranted, it may be said that, in general, the percentage of nitrogen taken up by the fused iron increases somewhat as the current density increases. With the lowest current densities used the amount of nitrogen was found to be appreciably less than for the higher current densities used. Paterson reports [3] an increase in nitrogen content under similar conditions.

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TABLE 4.—Relation Between Nitrogen Content and Current Density

Diameter of electrode	Approximate current	Current density	Nitrogen content[a]			
			Specimen A	Specimen B	Specimen C	Average
	Amperes	Amp./in.²	Per cent	Per cent	Per cent	Per cent
1/8 inch	110	9 000	0.156 [b] 0.149	0.152 [b] 0.141	.....	0.138
1/8 inch	145	11 800	.127 [b] .140	.132 [b] .135	.....	.126
5/32 inch	145	7 600	.140 [b] .121	.124 [b] .122	.....	.127
5/32 inch	185	9 650	.123 [b] .119	.121 [b,d] .132	.....	.131
5/32 inch	225	11 700	.124 [b] .113	.117 [b] .123	.....	[c]
5/32 inch	175	9 100	.....	.....	0.133 .098	[e]
3/16 inch	185	6 700	.126 [b] .127	.119 [b] .106	.....	.120
3/16 inch	225	8 150	.131 [b] .131	.111 [b] .108	.....	.120
3/16 inch	260	9 400	.133 [b] .134	.112 .094	.....	.118
5/16 inch	300	3 900	.117 [b] .111	.....	.....	.114

[a] Average of two determinations.
[b] Coated electrodes.
[c] Included in average for C–D, 11 800.
[d] Average of nine determinations.
[e] Included in average for C–D, 9000.

VI. MECHANICAL PROPERTIES OF THE ARC-FUSED METAL

The mechanical properties of the two types of electrodes used as determined by the tension test are summarized in Table 5 below.

TABLE 5.—Tensile Properties of Electrodes

Electrode type	Diameter	Proportional limit	Ultimate tensile strength	Elongation in 2 inches	Reduction of area
	Inch	Lbs./in.²	Lbs./in.²	Per cent	Per cent
A	5/32	39 000	65 800	16.5	69.2
A	3/16	48 000	62 100	9.0	69.3
A	5/16	34 000	60 100	14.0	66.4
A	5/16	.....	57 300	15.5	67.6
B	5/32	67 000	88 600	4.5	51.3
B	3/16	58 500	84 700	7.0	59.8
B	1/4	37 500	66 300	15.0	61.4
B	1/4	.....	67 900	15.5	62.4

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In Table 6 are given the results of the mechanical tests made upon the tension specimens which were turned out of the blocks of metal resulting from the fusion of the electrodes. For purposes of comparison the chemical composition of the material is also included in the table. The specimens listed C–1, C–2,. . . .C–6 are the six which were prepared outside the Bureau and submitted for purposes of comparison. It was stated that they were prepared from bare electrodes five thirty-seconds inch in diameter, of type B, containing 0.17 per cent carbon and 0.5 per cent manganese.

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TABLE 6.—Tensile Properties and Results of Chemical Analyses of 54 Specimens of Weld Metal (0.505 Inch Diameter Standard Tension Bar Used)

BARE ELECTRODES

Sample No.	Diameter of electrode	Direct current	Tensile properties			Elongation in 2 in.	Reduction of area	Hardness		C	Si	Mn	P	S	Cu	N_2
			Ultimate tensile strength	Yield point	Proportional limit			Brinell	Scleroscope							
	Inch	Amp.	Lbs./in.²	Lbs./in.²	Lbs./in.²	Per cent	Per cent									
A-2	1/8	110	49 850	36 600	25 000	6.0	6.5	108	17.0	.....	.....	.....	.....	.....	.....	0.156
A-3	1/8	145	51 950	36 250	30 000	8.0	13.0	114	18.0	0.046	0.007	Tr	0.005	0.036	.....	.127
A-7	5/32	145	47 550	.....	.....	6.0	7.4	108	15.6	.....	.....	.....	.....	.....	.....	.140
A-8	5/32	185	48 100	.....	.....	8.0	8.7	104	16.2	.....	.....	.....	.....	.....	.....	.123
A-9	5/32	225	45 500	.....	.....	8.0	9.6	101	15.2	.....	.....	.....	.....	.....	.....	.124
A-4	3/16	185	50 600	33 750	29 500	5.5	13.5	105	16.2	.010	.012	Tr	.003	.033	.....	.126
A-5	3/16	225	49 150	36 250	22 000	7.0	10.0	102	16.4	.033	.006	0.069	.012	.043	0.058	.131
A-6	3/16	260	50 950	33 750	28 800	10.5	12.0	107	17.4	.....	.....	.....	.....	.....	.....	.133
A-10	5/16	300	46 670	.....	.....	12.0	11.9	104	16.2	.050	.11	.014	.011	.026	.063	.117

COVERED ELECTRODES

AD-2	1/8	110	51 250	35 000	25 600	9.5	11.0	103	17.8	.....	.....	.....	.....	.....	.....	.149
AD-2[a]	1/8	110	43 000	.....	23 000	5.0	9.0	.....	.....	.....	.....	.....	.....	.....	.....	.....
AD-3	1/8	145	51 100	33 750	25 000	8.5	10.5	110	17.2	.031	.007	Tr	.005	.033	.....	.140
AD-3[a]	1/8	145	46 250	.....	24 250	7.0	12.0	.....	.....	.....	.....	.....	.....	.....	.....	.....
AD-7	5/32	145	41 750	.....	.....	6.0	6.6	99	17.4	.....	.....	.....	.....	.....	.....	.121
AD-7[a]	5/32	145	46 950	.....	25 500	8.0	9.4	.....	.....	.....	.....	.....	.....	.....	.....	.....
AD-8	5/32	185	44 620	.....	.....	6.5	5.8	103	16.8	.....	.....	.....	.....	.....	.....	.119
AD-8[a]	5/32	185	43 600	.....	23 250	6.5	9.0	.....	.....	.....	.....	.....	.....	.....	.....	.....
AD-9	5/32	225	46 900	.....	.....	9.5	10.1	96	16.8	.....	.....	.....	.....	.....	.....	.113
AD-9[a]	5/32	225	41 550	.....	25 500	5.0	6.5	.....	.....	.....	.....	.....	.....	.....	.....	.....
AD-4	3/16	185	51 200	35 000	30 000	10.5	10.5	101	17.2	.010	.014	Tr	.002	.035	.....	.127
AD-4[a]	3/16	185	45 700	.....	25 500	8.5	11.5	.....	.....	.....	.....	.....	.....	.....	.....	.....

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AD-5	3/16	225	48 600	35 000	30 000	7.0	10.0	96	16.2								.131
AD-5[a]	3/16	225	46 250		23 750	11.5	12.0										
AD-6	3/16	260	47 500	34 500	31 500	9.0	9.0	97	14.6								.134
AD-6[a]	3/16	260	50 700			8.0	2.8	105	13.6								
AD-10	5/16	300	45 900			8.5	11.5	98	15.4								.111
BARE ELECTRODES																	
B-2	1/8	110	52 650	37 000	27 000	7.5	7.5	114	18.8								[b].152
B-3	1/8	145	54 500	36 000	27 000	12.5	12.0	106	15.8	.027	.008	Tr	.002	.035			[b].132
B-4	5/32	145	46 450	33 500	26 000	5.0	7.0	102	16.2								[b].124
B-5	5/32	185	49 600	34 250	27 000	7.5	9.0	108	16.2								[b].121
B-6	5/32	225	49 500	30 500	28 000	9.0	7.5	110	16.6								[b].117
B-7	3/16	185	47 550		28 500	7.5	11.5	95	17.4								[b].119
B-8	5/32	225	42 900		18 750	7.5	16.2	101	16.2								[b].111
B-9	5/32	260	47 500		21 500	12.0	13.5	102	15.4								[b].112
COVERED ELECTRODES																	
BD-2	1/8	110	49 050	33 750	27 500	9.0	12.0	100	16.0								[b].141
BD-2[a]	1/8	110	44 400		20 000	6.5	9.4										
BD-3	1/8	145	52 100	34 300	30 500	12.5	16.0	116	16.0	.024	.010	Tr	.004	.035			[b].135
BD-3[a]	1/8	145	50 850		23 500	13.0	17.5										
BD-4	5/32	145	48 130	31 000	30 500	8.0	10.0	101	14.8								[b].122
BD-4[a]	5/32	145	41 750		21 000	6.0	9.5										
BD-5	5/32	185	49 086	31 730	29 000	12.5	13.0	97	16.0								[c].132
BD-5[a]	5/32	185	47 100		22 500	11.0	12.5										
BD-6	5/32	225	45 500	30 500	25 000	8.5	10.5	95	15.6								[b].123
BD-7	3/16	185	49 950		24 500	11.5	21.5	98	16.8								[b].106
BD-7[a]	3/16	185	51 150		23 750	14.5	19.5										
BD-8	3/16	225	41 500		17 850	6.0	12.7	99	13.8								[b].108
BD-8[a d]	[d]3/16	[d]225	48 750		21 250	12.5	16.0										
BD-9	3/16	260	46 350		24 000	10.0	15.0	99	14.6								[b].094

[a] Second test of duplicate specimen. [b] Average of two determinations. [c] Average of nine determinations. [d] Doubtful.

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TABLE 6.—Tensile Properties and Results of Chemical Analyses of 54 Specimens of Weld Metal (0.505 Inch Diameter Standard Tension Bar Used)—Continued

BARE ELECTRODES

Sample No.	Diameter of electrode	Direct current	Tensile properties			Elongation in 2 in.	Reduction of area	Hardness		C	Si	Mn	P	S	Cu	N_2
			Ultimate tensile strength	Yield point	Proportional limit			Brinell	Scleroscope							
	Inch	Amp.	Lbs./in.²	Lbs./in.²	Lbs./in.²	Per cent	Per cent									
C-1	5/32	175	48 650	32 650	23 000	12.0	19.1									
C-2	5/32	175	45 200	32 400	23 000	7.5	16.6									
C-3	5/32	175	49 720	32 650	25 000	9.0	13.6									
C-4	5/32	175	54 500	32 500	25 000	11.0	17.5	118	19.2							[b] 0.133
C-5	5/32	175	50 900	32 500	24 000	15,0	23.0	109	15.2							[b] 0.098
C-6	5/32	175	50 500	33 500	23 000	12.0	16.0									

[b] Average of two determinations.

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As an aid for more readily comparing the mechanical properties of the two types of arc-fused metal A and B, the results have been grouped as given in Table 7.

TABLE 7.—Tensile Properties and Hardness of Specimens of Weld Metal Prepared by the Bureau (Arranged in Order of Amperage Used)

Current used, amperes [a]		Tensile properties							
		Ultimate tensile strength				Yield point			
		Electrode A [b]		Electrode B [b]		Electrode A		Electrode B	
		Bare	Covered	Bare	Covered	Bare	Covered	Bare	Covered
		Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²
110		49 850	51 250	52 650	49 050	36 600	35 000	37 000	33 750
		.....	[c] 43 000	.....	[c] 44 400	.....	.....	.....	.....
145	Series 1	51 950	51 000	54 500	52 100	36 250	36 750	36 000	34 300
		.....	[c] 46 250	.....	[c] 50 850	.....	.....	.....	.....
	Series 2	47 550	41 750	46 450	48 130	.....	.....	33 500	31 000
		.....	[c] 46 950	.....	[c] 41 750	.....	.....	.....	.....
185	Series 1	48 100	44 620	49 600	49 086	.....	.....	34 250	31 730
		.....	[c] 43 600	.....	[c] 47 100	.....	.....	.....	.....
	Series 2	50 600	51 200	47 550	49 950	33 750	35 000	.....	.....
		.....	[c] 45 700	.....	[c] 51 150	.....	.....	.....	.....
225	Series 1	45 500	46 900	49 500	45 500	.....	.....	30 500	30 500
		.....	[c] 41 550	.....	.....	.....	.....	.....	.....
	Series 2	49 150	48 600	42 900	41 500	36 250	35 000	.....	.....
		.....	[c] 46 250	.....	.....	.....	.....	.....	.....
260		50 950	47 500	47 500	46 350	33 750	34 500	.....	.....
		.....	[c] 50 700	.....	.....	.....	.....	.....	.....
300		46 670	45 900	.....	.....	.....	.....	.....	.....
Average		48 900	46 600	48 800	47 450	35 300	34 650	34 250	32 250
		47 400		47 980		35 000		33 250	

[a] Size of electrode used: One-eighth inch diameter, 110 amperes and 145 amperes (series 1); 5/32 inch diameter, 145 amperes (series 2), 185 amperes (series 1), and 225 amperes (series 1); 3/16 inch diameter, 185 amperes (series 2), 225 amperes (series 2), and 260 amperes; 5/16 inch diameter, 300 amperes.

[b] A and B refer to the two types of electrodes used (Table 3).

[c] Duplicate specimen.

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TABLE 7.—Tensile Properties and Hardness of Specimens of Weld Metal Prepared by the Bureau (Arranged in Order of Amperage Used)—Continued

Current used, amperes	Tensile properties—Continued							
	Elongation in 2 inches				Reduction in area			
	Electrode A		Electrode B		Electrode A		Electrode B	
	Bare	Covered	Bare	Covered	Bare	Covered	Bare	Covered
	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
110	6.0	9.5	7.5	9.0	6.5	11.0	7.5	12.0
	.....	[a] 5.0	.....	[a] 6.5	.....	[a] 9.0	.....	[a] 9.4
145, Series 1	8.0	8.5	12.5	12.5	13.0	10.5	12.0	16.0
	.....	[a] 7.0	.....	[a] 13.0	.....	[a] 12.0	.....	[a] 17.5
145, Series 2	6.0	6.0	5.0	8.0	7.4	6.6	7.0	10.0
	.....	[a] 8.0	.....	[a] 6.0	.....	[a] 9.4	.....	[a] 9.5
185, Series 1	8.0	6.5	7.5	12.5	8.7	5.8	9.0	13.0
	.....	[a] 6.5	.....	[a] 11.0	.....	[a] 9.0	.....	[a] 12.5
185, Series 2	5.5	10.5	7.5	11.5	23.5	10.5	11.5	21.5
	.....	[a] 8.5	.....	[a] 14.5	.....	[a] 11.5	.....	[a] 19.5
225, Series 1	8.0	9.5	9.0	8.5	9.6	10.1	7.5	10.5
	.....	[a] 5.0	.....	.....	.....	[a] 6.5	.....	.....
225, Series 2	7.0	7.0	7.5	6.0	10.0	10.0	16.2	12.7
	.....	[a] 11.5	.....	.....	.....	[a] 12.0	.....	.....
260	10.5	9.0	12.0	10.0	12.0	9.0	13.5	15.0
	.....	[a] 8.0	.....	.....	.....	[a] 2.8	.....	.....
300	12.0	8.5	.....	.....	11.9	11.5	.....	.....
Average	7.9	8.5	8.5	9.9	10.3	9.2	10.5	13.8
	7.9		9.0		9.6		12.5	

Current used, amperes	Brinell and scleroscope hardness							
	Electrode A				Electrode B			
	Brinell		Scleroscope		Brinell		Scleroscope	
	Bare	Covered	Bare	Covered	Bare	Covered	Bare	Covered
110	108	103	17.0	17.8	114	100	18.8	16.0
	.....	.....	.....	.....	.....	.....	.....	.....
145, Series 1	114	110	18.0	17.2	106	116	15.8	16.0
	.....	.....	.....	.....	.....	.....	.....	.....
145, Series 2	108	99	15.6	17.4	102	101	16.2	14.8
	.....	.....	.....	.....	.....	.....	.....	.....
185, Series 1	104	103	16.2	16.8	108	97	16.2	16.0
	.....	.....	.....	.....	.....	.....	.....	.....
185, Series 2	105	101	16.2	17.2	95	98	17.4	16.8
	.....	.....	.....	.....	.....	.....	.....	.....
225, Series 1	101	96	15.2	16.8	110	95	16.6	15.6
	.....	.....	.....	.....	.....	.....	.....	.....
225, Series 2	102	96	16.4	16.2	101	99	16.2	13.8
	.....	.....	.....	.....	.....	.....	.....	.....
260	107	97	17.4	14.6	102	99	15.4	14.6
	.....	.....	.....	.....	.....	.....	.....	.....
300	104	.....	16.2	.....	.....	.....	.....	.....
Average	106	100	16.5	16.8	105	101	16.6	15.5
	103		16.6		103		16.0	

[a] Duplicate specimen.

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The characteristic appearance of specimens after testing, illustrating their behavior when stressed in tension until rupture occurs, is shown in Figs. 5, *a* and *b*. These represent two views

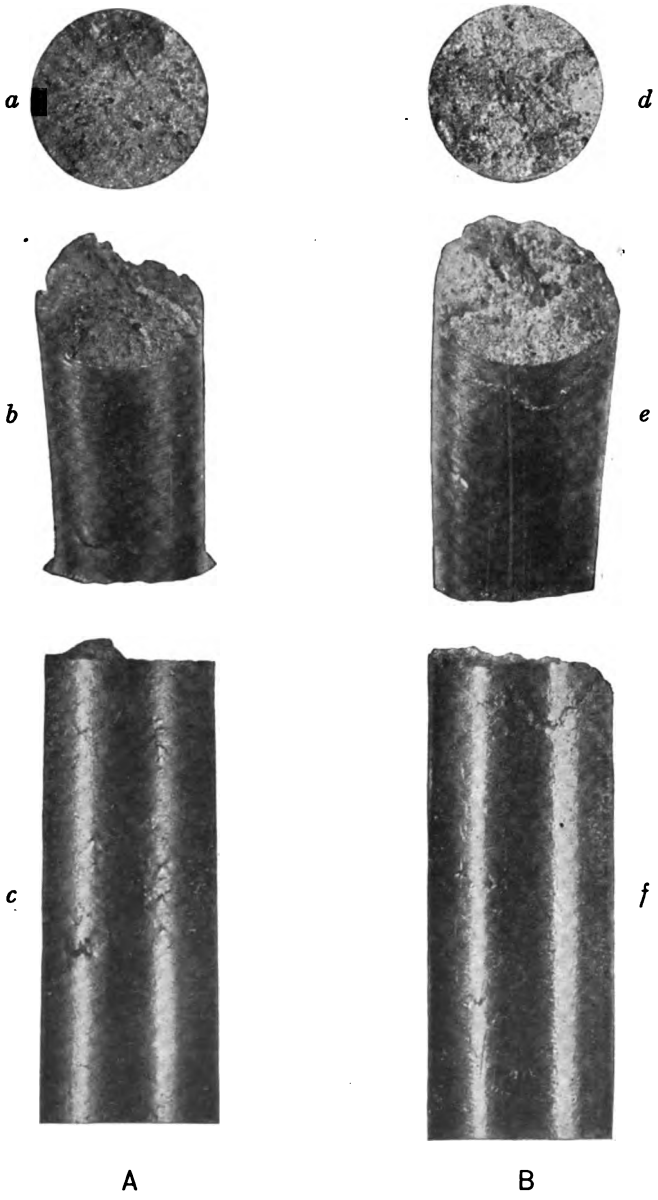

FIG. 5.—*Characteristic appearance of tension specimens after test.* × 2

A, specimen B-3 (Table 6); *B*, specimen BD-6 (Table 6); *a*, *d*, face of fracture, viewed normally; *b*, *e*, fractured end of specimen, viewed at an angle of 45°; *c*, *f*, side view of specimen

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of the face of the fracture, one in which the line of vision is perpendicular to the face, the other at an angle of 45°, together with a side view of the cylindrical surface of the specimen. The features shown in Fig. 5 are characteristic of all the specimens tested, though in some they were much more pronounced than those shown. The fracture of the specimen in all cases reveals interior flaws. In some of the specimens, however, these are microscopic and of the character shown in Fig. 12. Although many of the specimens (from the results of Table 6) appear to have a considerable elongation, it is seen from Fig. 5 that the measured elongation does not truly represent a property of the metal itself. It is due rather to interior defects which indicate lack of perfect union of succeeding additions of metal during the process of fusion. The surface markings of the specimen after stressing to rupture are very similar to those seen in the familiar "flaky steel."

It appears from the results above that, as far as the mechanical properties are concerned, nothing was gained by coating the electrodes. The results show no decided superiority for either of the two types of electrodes used. This may be expected, however, when one considers that the two are rendered practically the same in composition during fusion by the burning out of the carbon and other elements.

The results of the tension tests upon the C series of specimens, which were made outside of this Bureau and submitted to be included in the investigation, show no marked difference between these samples and those prepared by this Bureau. In all cases the results obtained in the tension test are determined by the soundness of the metal and do not necessarily indicate the real mechanical properties of the material.

The results of the hardness determinations do not appear to have any particular or unusual significance. The variations are of the same general nature and relative magnitude as the variations observed in the results of the tension test. In general, the higher hardness number accompanies the higher tensile values, though this was not invariably so. As previously noted, specimens were prepared for the purpose of showing the relation between the direction in which the stress is applied and the manner of deposition of the metal. The metal was deposited in the form shown in Fig. 3, except that the "beads" extended across the piece rather than lengthwise, hence the "beads" of fused metal were at right angles to the direction in which the tensional stress

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was applied. The results of the tension tests show that these two specimens (AW–1 and AW–2) were decidedly inferior to those prepared in the other manner, as shown in the table below.

TABLE 8.—Mechanical Properties of Arc-Fused Metal Deposited at Right Angles to Length of Specimen

Specimen	Proportional limit	Ultimate tensile strength	Elongation in 2 inches	Reduction of area
	Lbs./in.²	Lbs./in.²	Per cent	Per cent
AW–1	22 500	40 450	6.5	8.5
AW–2	22 500	39 500	4.0	3.0

VII. STRUCTURAL FEATURES: METALLOGRAPHIC EXAMINATION

1. MACROSTRUCTURE

The general condition of the metal resulting from the arc fusion is shown in Figs. 6 and 7, which show longitudinal medial sections of a series of the tension bars adjacent to the fractured end. The metal in all of these specimens was found to contain a considerable number of cavities and oxide inclusions; these are best seen after the surfaces are etched with a 10 per cent aqueous solution of copper-ammonium chloride. In many of the specimens the successive additions of metal are outlined by a series of very fine inclusions (probably oxide) which are revealed by the etching. There appears to be no definite relation between the soundness of the metal and the conditions of deposition—that is, for the range of current density used—nor does either type of electrode used show any decided superiority over the other with respect to porosity of the resulting fusion. In Fig. 8, *a* and *b*, is shown the appearance of a cross section of one of the blocks of arc-fused metal prepared outside of this Bureau by skilled welding operators. The condition of this material is quite similar to that prepared by this Bureau. The appearance of the fracture of one of the tension specimens made from such material is shown in Fig. 8*c*. The same defects as may be seen in the cross sections (Fig. 8, *a* and *b*) are also to be found on the fractured end of the bar; in addition to these, there appears to be a seam of unwelded metal.

The later microscopic study of the material also revealed further evidence of unsoundness in all the three types, A, B, and C.

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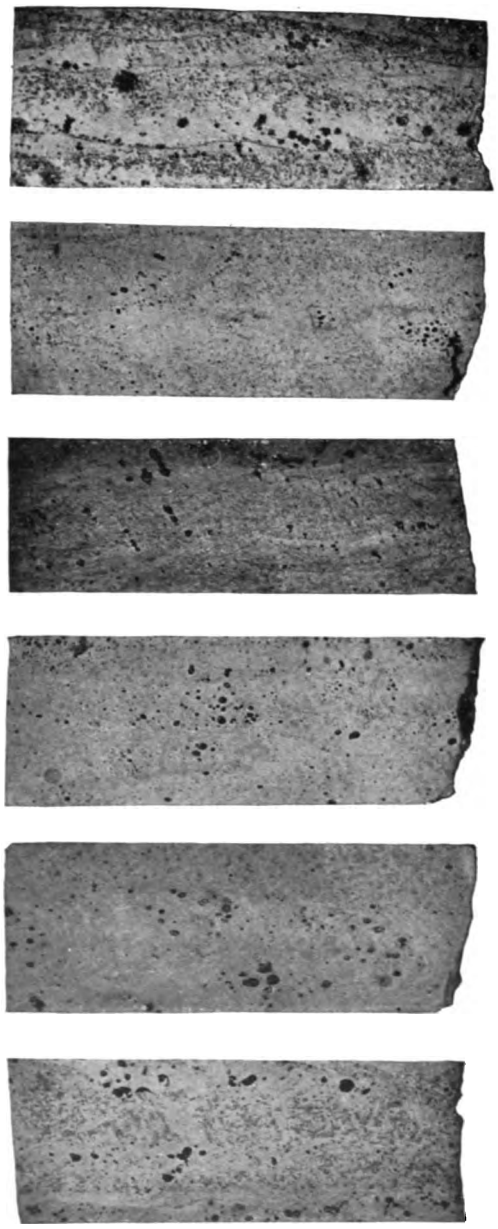

FIG. 6.—*Macrostructure of arc-fused metal from electrodes of type A.* × 2

Medial longitudinal sections of tension specimens (Table 6) were etched with 10 per cent aqueous solution of copper ammonium chloride. *a*, specimen AD–6, $\frac{3}{16}$-inch covered electrode, 260 amperes; *b*, specimen A–5, $\frac{3}{16}$-inch bare electrode, 225 amperes; *c*, specimen A–6, $\frac{3}{16}$-inch bare electrode, 260 amperes; *d*, specimen A–3, $\frac{1}{8}$-inch bare electrode, 145 amperes; *e*, specimen A–4, $\frac{3}{16}$-inch bare electrode, 185 amperes; *f*, specimen AD–2, $\frac{1}{8}$-inch covered electrode, 110 amperes

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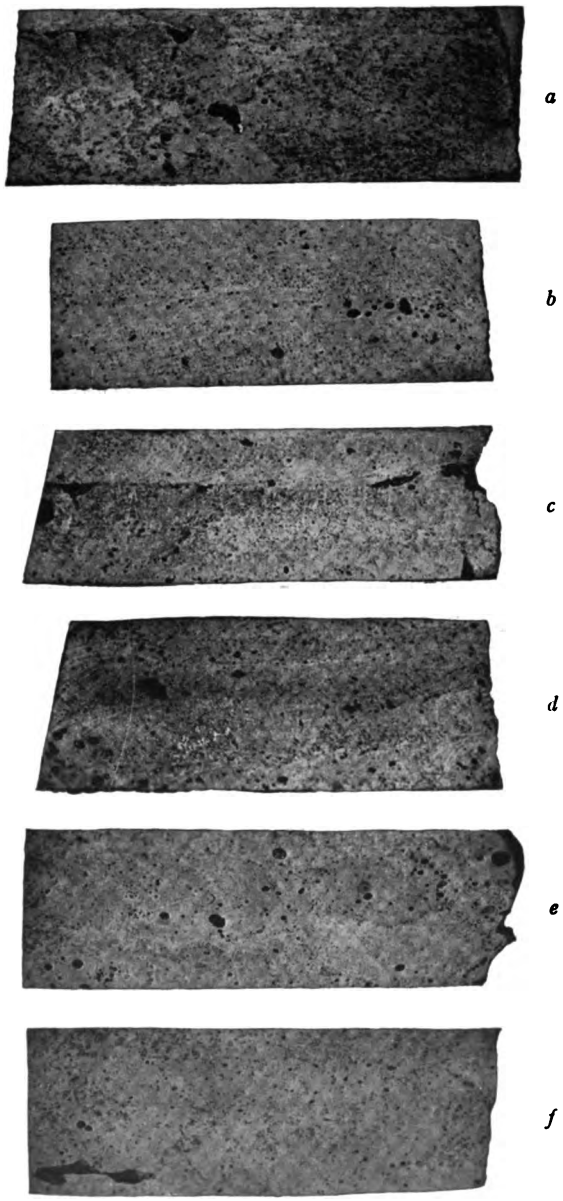

FIG. 7.—*Macrostructure of arc-fused metal from electrodes of type B.* × 2

Medial longitudinal sections of tension specimens (Table 6) were etched with 10 per cent aqueous solution of copper ammonium chloride. *a*, specimen B–4, $\frac{1}{4}$-inch bare electrode, 145 amperes; *b*, specimen B–5, $\frac{1}{4}$-inch bare electrode, 185 amperes; *c*, specimen B–3, $\frac{1}{8}$-inch bare electrode, 110 amperes; *d*, specimen B–3, $\frac{1}{8}$-inch bare electrode, 145 amperes; *e*, specimen BD–6, $\frac{1}{4}$-inch covered electrode, 225 amperes; *f*, specimen BD–4, $\frac{1}{4}$-inch covered electrode, 145 amperes

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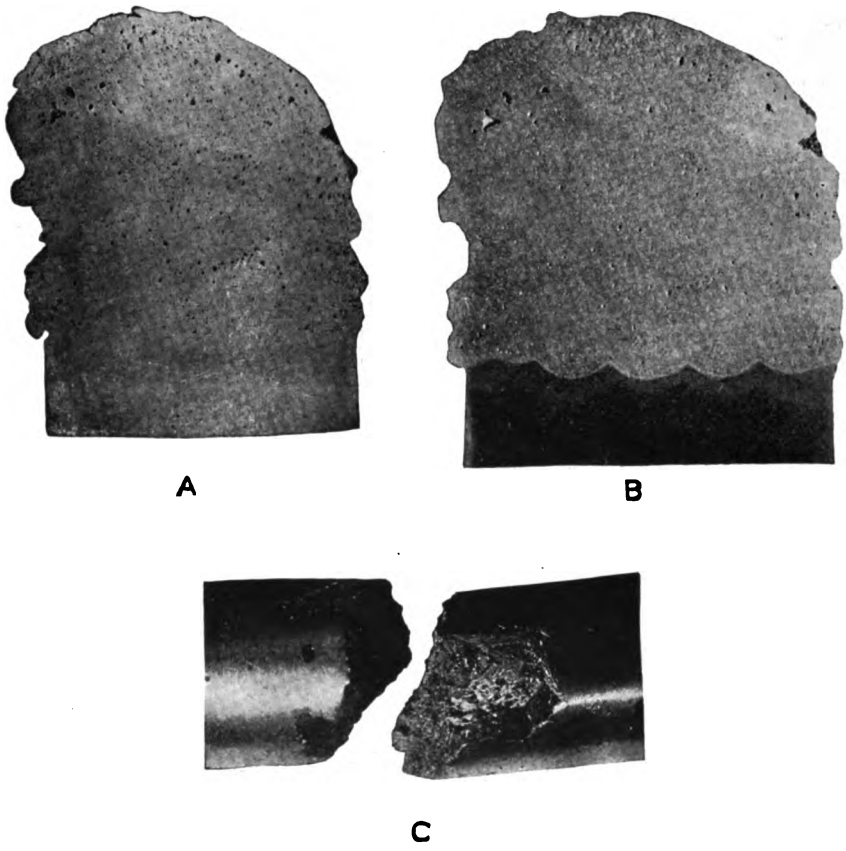

FIG. 8.—*Macrostructure of arc-fused metal, Type C*

A, cross section of the block of arc-fused metal from which specimen C-1 (Table 6) was turned; unetched, × 1; *B*, same as *A*, etched with 10 per cent aqueous solution of copper ammonium chloride, × 1.2; *C*, appearance of fractured tension specimen C-4 (Table 6), × 2

2. MICROSTRUCTURE

(*a*) GENERAL FEATURES.—For purposes of comparison the microstructure of the electrodes before fusion is shown in FIG. 9. The A electrodes have the appearance of steel of a very low carbon content. In some cases they were in the cold-rolled state; all showed a considerable number of inclusions. The B electrodes have the structure of a mild steel and are much freer from inclusions than are those of the other type. It is undoubtedly true, however, that the condition of the arc-fused metal with respect to the number of inclusions is a result of the fusion rather than of the initial state of the metal.

It is to be expected that the microstructure of the material after fusion will be very considerably changed, since the metal

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is then essentially the same as a casting. It has some features, however, which are not to be found in steel as ordinarily cast.

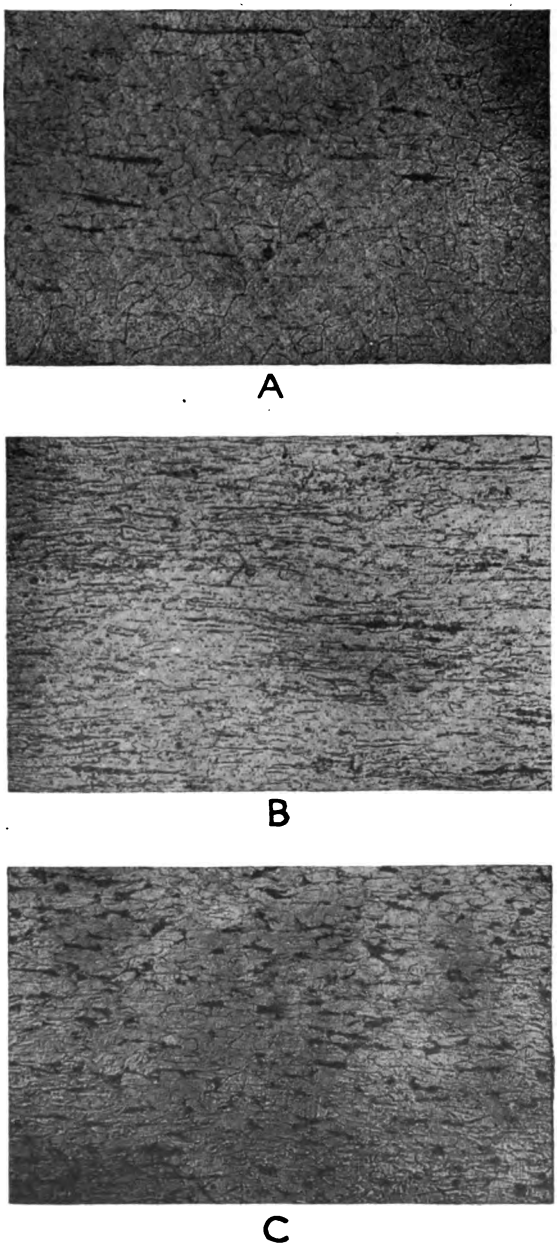

FIG. 9.—*Microstructure of electrodes before fusion.* × 100

A, electrode of type A, ⅛ inch, annealed as received; *B*, electrode of type B, ¼ inch, cold drawn as received; *C*, electrode of type B, ⅛ inch, annealed as received. Etching reagent, 5 per cent alcoholic solution of picric acid

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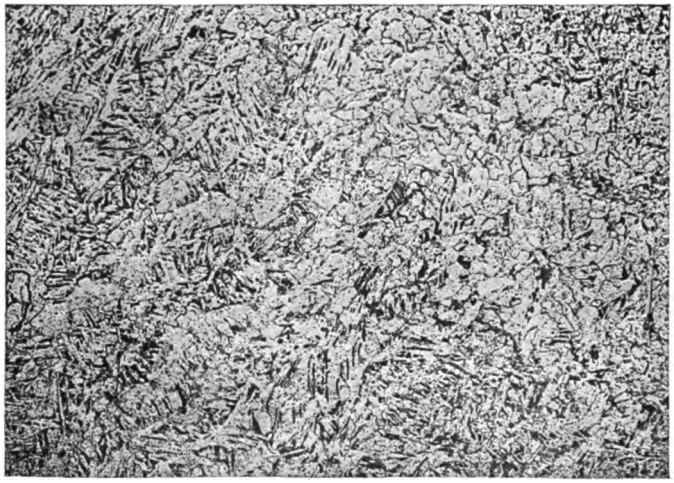
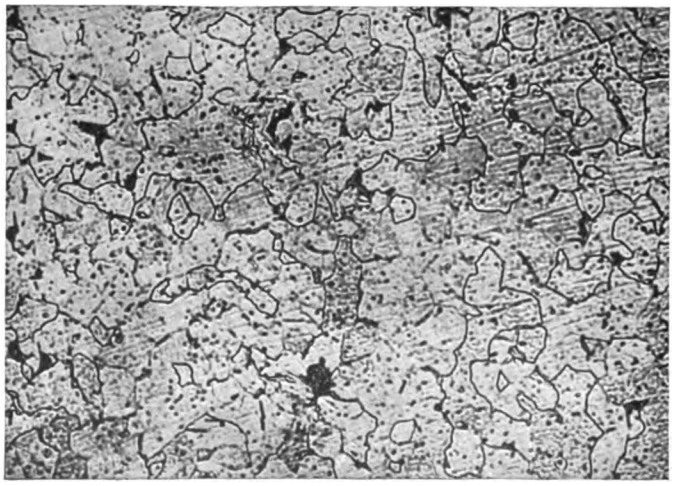
The general type of microstructure was found to vary in the different specimens and to range from a condition which will be designated as "columnar" to that of a uniform fine equi-axed



A



B

FIG. 10.—*Typical microstructures of arc-fused metal, designated as "columnar" and "equiaxed."* × 100

A, columnar condition (specimen B–2, Table 6); etching reagents, 5 per cent alcoholic solution of picric acid; an equiaxed condition is shown in the upper right-hand corner; *B*, equiaxed condition (specimen AD–3, Table 6). Etching reagent, 2 per cent alcoholic solution of nitric acid

crystalline arrangement. This observation held true for both types of electrodes, whether bare or covered. Fig. 10 shows the

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two extremes of microstructure which were found. In the examination of cross sections of the blocks of arc-fused metal, it was noticed that the equi-axed type of structure is prevalent throughout the interior of the piece and the columnar is to be found generally nearer the surface; that is, in the metal deposited last. It may be inferred from this that the metal of the layers which were deposited during the early part of the preparation of the specimen is refined considerably by the successive heatings to which it is subjected as additional layers of metal are deposited.

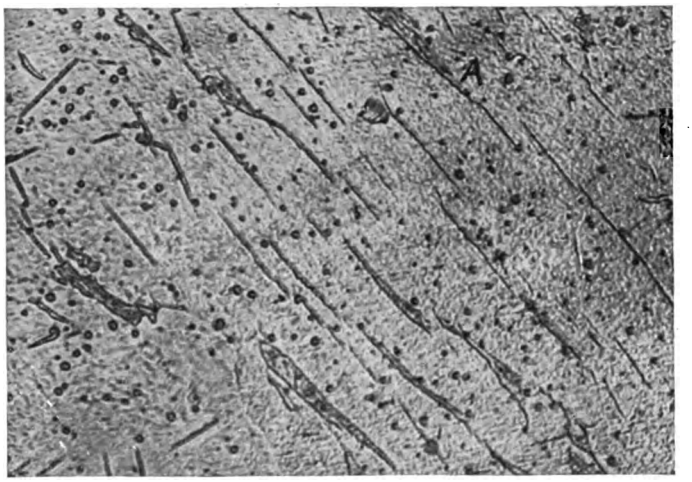

FIG. 11.—*Microstructure of arc-fused steel showing characteristic features, pearlite islands, "needles," and oxide inclusions.* × 750

The specimen (AD–2, Table 6) was etched with 2 per cent alcoholic solution of nitric acid

The general type of structure of the tension bars cut from the blocks of arc-fused metal will vary considerably according to the amount of refining which has taken place as well as the relative position of the tension specimen within the block. In addition it was noticed that the columnar and coarse equi-axed crystalline condition appears to predominate with fusion at high current densities.

(*b*) MICROSCOPIC EVIDENCE OF UNSOUNDNESS.—In all of the specimens of arc-fused metal examined microscopically there appear to be numerous tiny globules, apparently oxide, as shown in Figs. 10, 11, 12, and 13. A magnification of 500 diameters is usually necessary to show these inclusions. In general, they appear to have no definite arrangement but occur indiscriminately throughout the crystals of iron.

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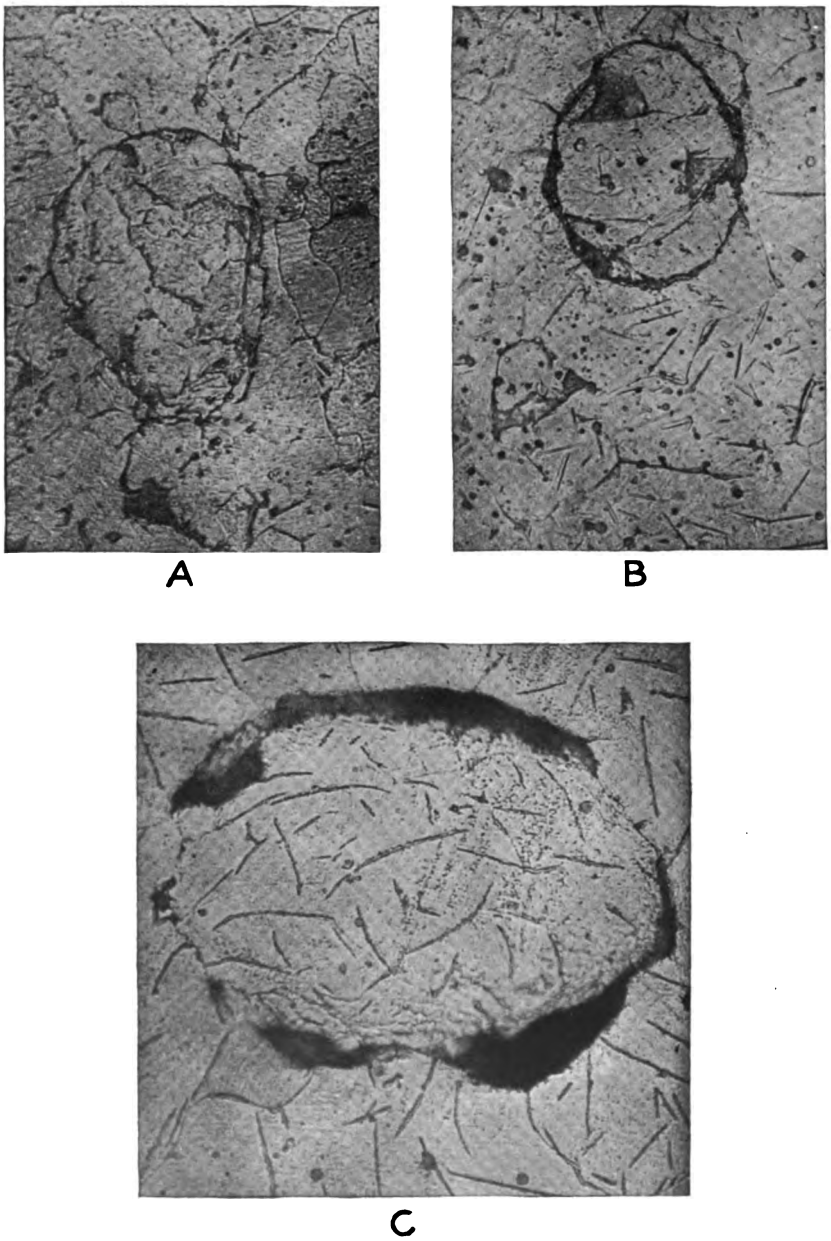

FIG. 12.—*Microstructure of arc-fused steel showing characteristic feature of "metallic-globule" inclusions.* × 500

A, specimen AD–9 (Table 6); *B*, specimen AD–10 (Table 6); *C*, specimen A–10 (Table 6) after heating 6 hours in vacuo. Etching reagent, 2 per cent alcoholic solution of nitric acid

A type of unsoundness frequently found is that shown in Fig. 12; this will be referred to as "metallic-globule inclusions." In

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general, these globules possess a microstructure similar to that of the surrounding metal, but are enveloped by a film, presumably of oxide. It seems probable that they are small metallic particles which were formed as a sort of spray at the tip of the electrode and which were deposited on the solidified crust surrounding the pool of molten metal directly under the arc. These solidified particles apparently are not fused in with the metal which is subsequently deposited over them; that is, during the formation of this same layer and before any brushing of the surface occurs. By taking extreme precautions during the fusion a great deal of the unsoundness may be avoided, and the mechanical properties of the metal may be considerably improved. However, the specimens as described are more representative of actual present practice in welding.

(*c*) CHARACTERISTIC "NEEDLES" OR "PLATES."—The most characteristic feature of the steel after fusion is the presence of numerous lines or needles within the crystals. The general appearance of this feature of the structure is shown in Fig. 13. The number and the distribution of these "needles" were found to vary greatly in the different specimens. They are most abundant in the columnar and in the coarse equi-axed crystals. The finer equi-axed crystals in some specimens were found to be quite free from them, although exceptions were found to this rule. In general, a needle lies entirely within the bounds of an individual crystal. Some instances were found, however, where a needle appears to lie across the boundary and so lies within two adjacent crystals. Several instances of this tendency have been noted in the literature on this subject ([12]—Fig. 8; [27]—Fig. 30; [18]—p. 251, Fig. 14 B). The needles have an appreciable width, and, when the specimen is etched with 2 per cent alcoholic nitric acid, they appear much the same as cementite; that is, they remain uncolored, although they may appear to widen and darken if the etching is prolonged considerably. The apparent widening is evidently due to the attack of the adjacent ferrite along the boundary line between the two. The tendency of the needles to darken when etched with a hot alkaline solution of sodium picrate as reported by Comstock [12] was confirmed. Fig. 13, *c* illustrates the appearance when etched in this manner. The needles are sometimes found in a rectangular grouping; that is, they form angles of 90° with one another. In other cases they appear to be arranged along the octahedral planes of the crystal;

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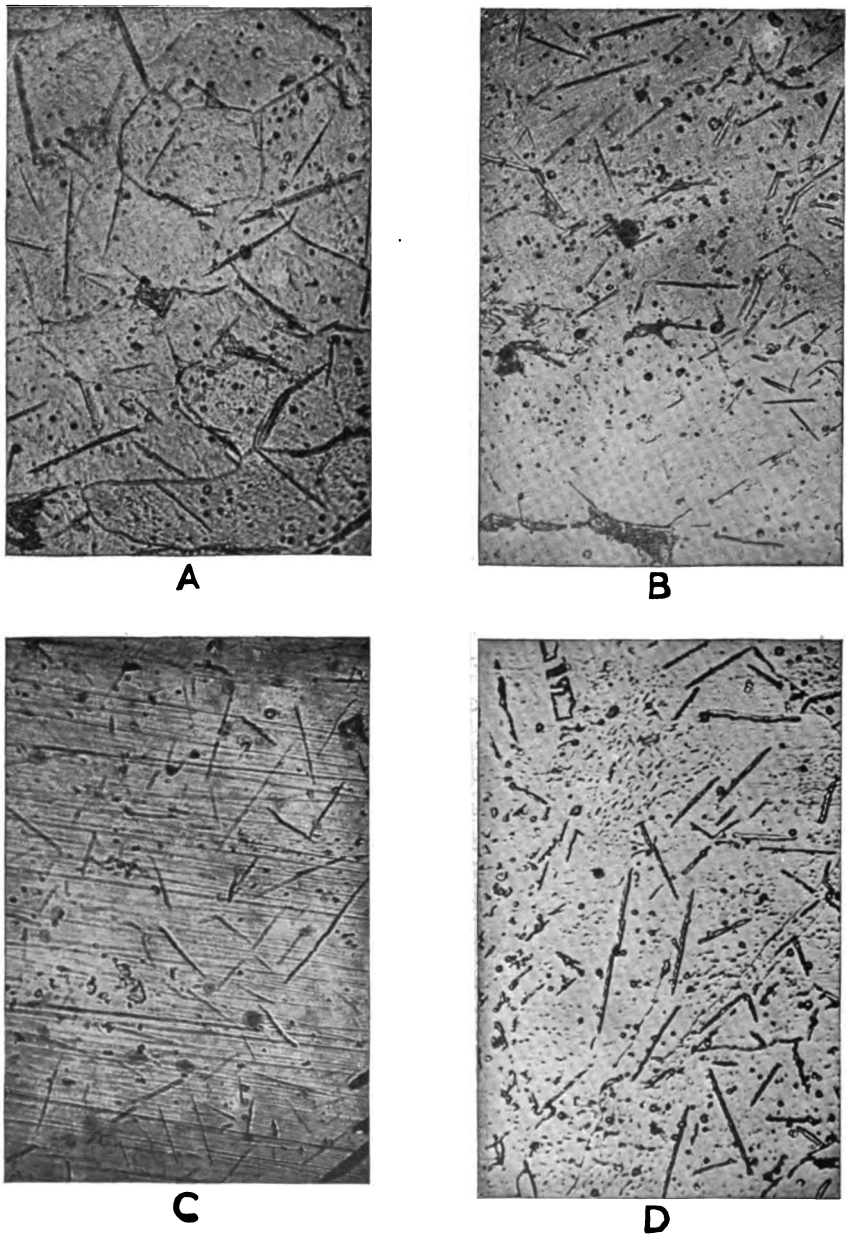

FIG. 13.—*Microstructure of arc-fused steel showing characteristic "needles" or "plates."*
× 500

A, specimen BD (Table 6); *B*, specimen BD–3 (Table 6); *C*, specimen A–5 (Table 6) (the material was used for thermal analysis (Sec. VII, 4) and was heated to approximately 900 ° C four times and cooled in vacuo); *D*, specimen of welded ship plate, one of the specimens used in the preliminary examinations (sec. 3); in addition to the usual needles a second generation of very small ones may be seen. Etching reagents: *A* and *B*, 5 per cent alcoholic solution of picric acid; *C*, hot alkaline solution of sodium picrate; *D*, 2 per cent alcoholic solution of nitric acid

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that is, at 60° to one another. This is best seen in specimens which have been heated as explained in section 3 below.

In some of the specimens certain crystals showed groups of very fine short needles (Fig. 13*d*). The needles comprising any one group or family are usually arranged parallel to one another, but the various groups are often arranged definitely with respect to one another in the same manner as described above. Similar needles have been reported in the literature by Miller [14].

An attempt was made [5] to determine whether the so-called lines or needles were really of the shape of needles or of tiny plates or scales. An area was carefully located on a specimen prepared for microscopic examination, which was then ground down slightly and repolished several times. It was possible to measure the amount of metal removed during the slight grinding by observing the gradual disappearance of certain of the spherical oxide inclusions the diameter of which could be accurately measured. By slightly etching the specimen after polishing anew it was possible to follow the gradual disappearance of some of the most prominent needles and to measure the maximum "depth" of such needles. It was concluded from the series of examinations that the term "plate" is more correctly descriptive of this feature of the structure than "line" or "needle." The thickness of the plate—that is, the width of the needle—varies from 0.0005 to 0.001 mm, and the width of the plate ("depth") may be as great as 0.005 mm. The persistence of the plates after a regrinding of the surface used for microscopical examination may be noted in some of the micrographs given by Miller [21]. The authors are not aware, however, of any other attempt to determine the shape of these plates by actual measurements of their dimensions.

The usual explanation of the nature of these plates is that they are due to nitrogen which is taken up by the iron during its fusion. Other suggestions which have been offered previously attribute them to oxide of iron and to carbide. The suggestion concerning oxide may be dismissed with a few words. The plates are distinctly different from oxide in their form and their behavior upon heating. It is shown later (sec. 3) that the tiny oxide globules coalesce into larger ones upon prolonged heating in vacuo, the plates also increase in size and become much more distinct (Fig. 25, *b*, *d*, *f*). In no case, however, was any intermediate stage

[5] This was carried out by P. D. Merica, formerly physicist of the Bureau of Standards.

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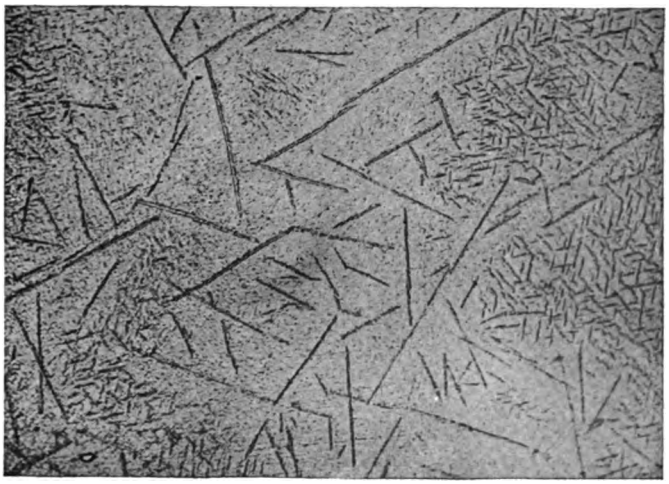
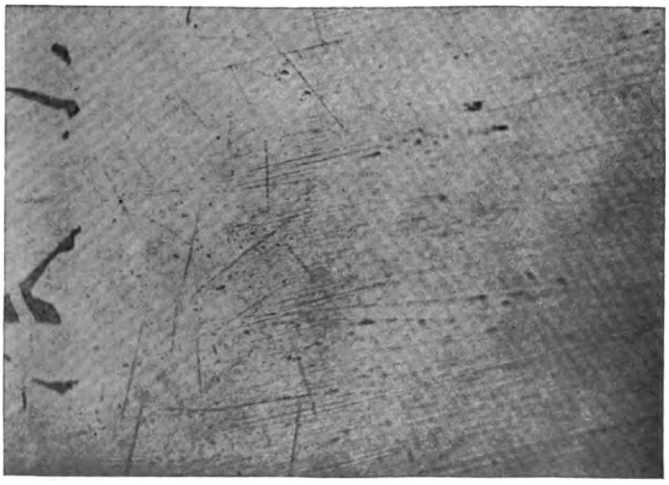
between the globular form and the plate produced such as would be expected if both were of the same chemical nature.



A



B

FIG. 14.—*Characteristic microstructure of "nitrogenized" iron.* × 500

Electrolytic iron was heated for some hours at approximately 650° C in ammonia gas. It takes up nitrogen readily under these conditions. *A*, two types of needles occur, etching reagent, 2 per cent alcoholic nitric acid; *B*, same specimen as *A* (another area). Etching reagent, hot alkaline solution of sodium picrate

Regarding the assumption that they are cementite plates, it may be said that the tendency during fusion is for the carbon to be burnt out, thus leaving an iron of low carbon content. In all the

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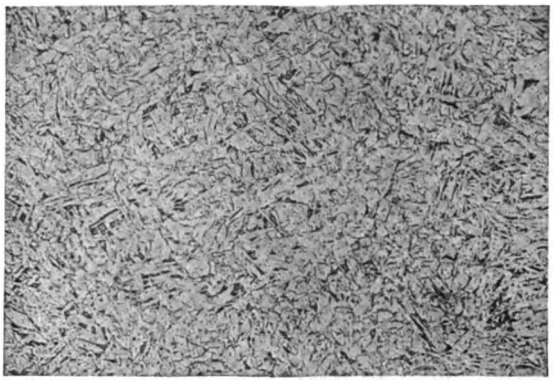
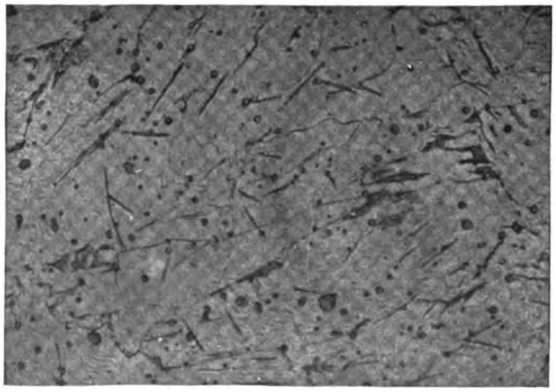
specimens islands of pearlite (usually with cementite borders) are to be found and may easily be distinguished with certainty. The number of such islands in any specimen appears to be sufficient to



A



B

FIG. 15.—*Microstructure of arc-fused steel produced in an atmosphere of carbon dioxide*

The structure is quite similar to that of the same fused in air; ¼-inch electrode of type A was used with 150 amperes. Etching reagent, 5 per cent alcoholic solution of picric acid; magnification, *A*, × 100; *B*, × 500

account for the carbon content of the material as revealed by chemical analysis. In some cases the pearlite islands are associated with a certain type of lines or needles such as are shown in Fig. 11. These needles, however, appear distinctly different from those of the prevailing type and are usually easily distinguished from them. The thermal analysis of arc-fused iron (Sec. VII, 4)

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gives further evidence that the plates are not to be attributed to carbon.

The fact that the plates found in the arc-fused metal are identical in appearance and in behavior (for example, etching) as those found in iron which has been nitrogenized is strong evidence that both are of the same nature. Fig. 14*a* shows the appearance of the plates produced in electrolytic iron by heating it for some time in pure ammonia gas. These plates behave in the same characteristic manner when etched with sodium picrate as do those occurring in arc-fused iron (Fig. 14*b*). The fact that the nitrogen content of the steel is increased by the arc fusion also supports the view that the change which occurs in the structure is due to the nitrogen. The statement has been made by Ruder [24] that metal fused in the absence of nitrogen—that is, in an atmosphere of carbon dioxide or of hydrogen—does not contain any plates, and hence the view that the plates are due to the nitrogen is very much strengthened. In Fig. 15 the appearance of specimens prepared at this Bureau by arc fusion of electrodes of type A in an atmosphere of carbon dioxide is shown. The microscopic examination of the fused metal shows unmistakable evidence of the presence of some plates, although they differ somewhat from those found in nitrogenized iron and in metal fused in the air by the electric arc. Evidently they are due to a different cause from the majority of those formed in the iron fused in air. For convenience, in the remainder of the discussion the plates in the arc-fused metal will be referred to as "nitride plates."

(*d*) Relation of Microstructure to the Path of Rupture.—The faces of the fracture of several of the tension specimens after testing were heavily plated electrolytically with copper, so as to preserve the edges of the specimens during the polishing of the section, and examined microscopically to see if the course of the path of rupture had been influenced to an appreciable extent by the microstructural features. In general, the fracture appears to be intercrystalline in type. Along the path of rupture in all of the specimens were smooth-edged hollows, many of which had evidently been occupied by the "metallic globules" referred to above, while others were gas holes or pores. Portions of the fracture were intracrystalline and presented a jagged outline, but it can not be stated with certainty whether the needles have influenced the break at such points or not. Fig. 16 shows the appearance of some of the fractures and illus-

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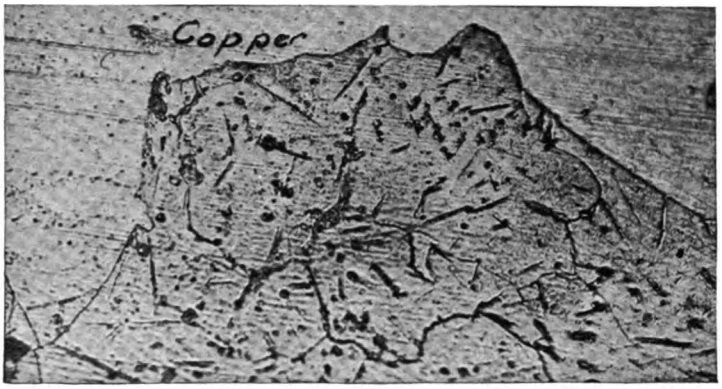
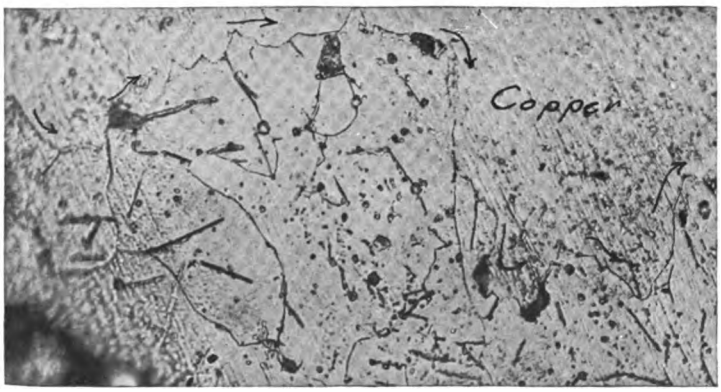


A

B



C

FIG. 16.—*Microstructure of arc-fused steel showing the relation of the path of rupture produced in tension to structural features.* × 500

A, Specimen B-4 (Table 6); *B*, same specimen as *A*, another spot; *C*, specimen B-6 (Table 6). The layer of copper indicated was deposited electrolytically on the fractured face of the specimen to protect it during polishing. Etching reagent, 2 per cent alcoholic solution of nitric acid

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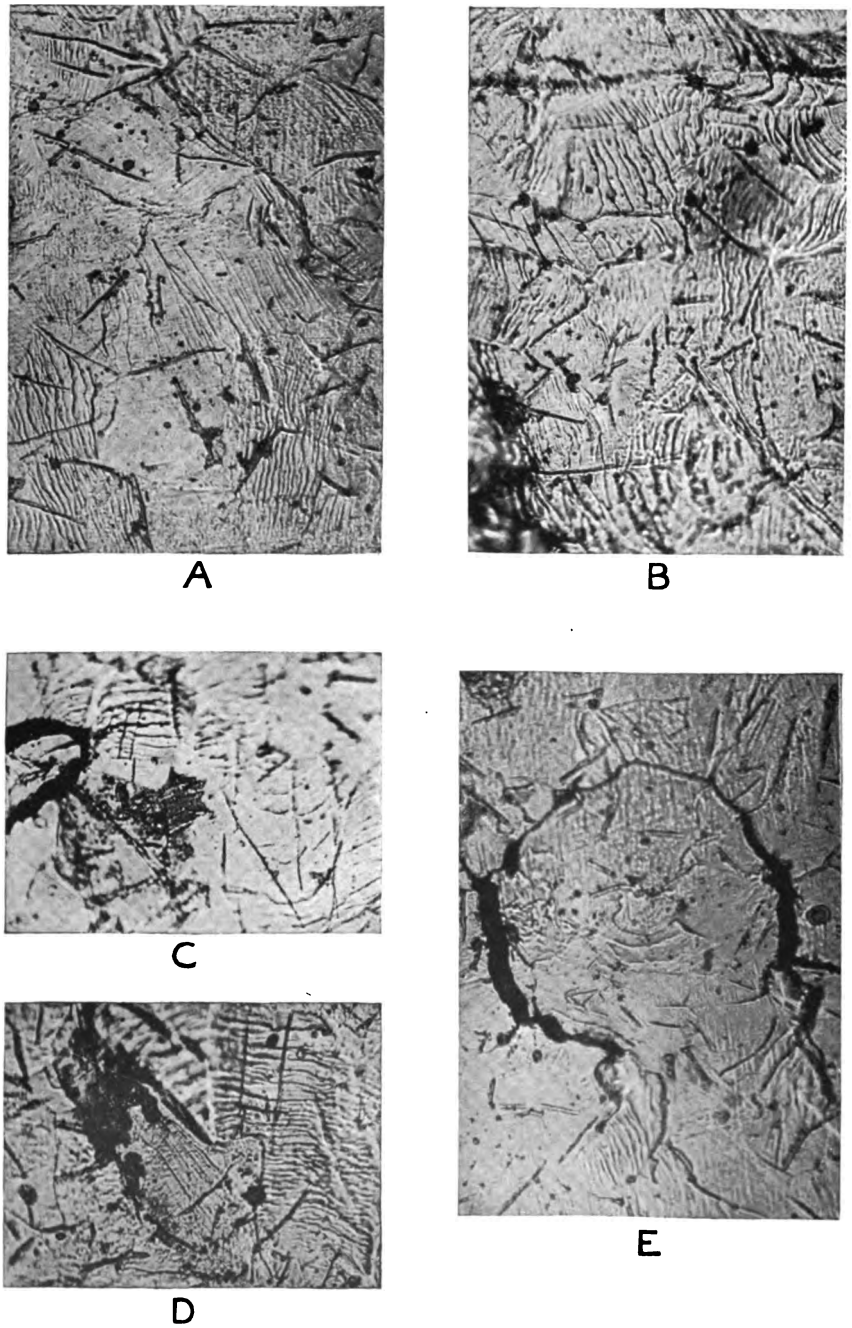

FIG. 17.—*Microstructure of arc-fused iron showing the behavior of "nitride plates" during plastic deformation of the metal.* × 500

A thin slice of specimen BD–2 (Table 6) was bent after polishing and etching with 2 per cent alcoholic nitric acid through an angle of 20° (approximate). *A* and *B* show that the "plates" have an effect upon the course of the slip bands similar to that of grain boundaries; that is, they cause a change of their direction; *C* and *D* show that a "faulting" of the "plates" sometimes occurs in grains which are very severely distorted; *E* shows that fracture upon bending usually starts in the oxide film inclosing "metallic globules" or in similar unsound areas

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trates that, in general, the nitride plates do not appear to determine to any appreciable extent the course of the path of rupture.

The behavior of the plates under deformation can best be seen in thin specimens of the metal which were bent through a considerable angle. Results of examination of welds treated in this manner have been described by Miller [14]. Small rectangular plates of the arc-fused metal, approximately three thirty-seconds of an inch thick, were polished and etched for microscopic examination and were then bent in the vise through an angle of 20° (approximate).

In Fig. 17 are given micrographs illustrating the characteristic behavior of the material when subjected to bending. For moderate distortion the nitride plates influence the course of the slip bands in much the same way that grain boundaries do; that is, the slip bands terminate usually on meeting one of the plates with a change of direction so that they form a sharper angle with the plate than does the portion of the slip band which is at some distance away. When the deformation is greater, the slip bands occur on both sides of the nitride plate, but usually show a slight variation in direction on the two sides of the nitride plate; this is often quite pronounced at the point where the plate is crossed by the slip band. In a few cases evidence of the "faulting" of the plate as a result of severe distortion was noted. This was rare, however, because of the nature of the metal. On account of the inclusions and other features of unsoundness of the metal, rupture occurs at such points before the sound crystals have been sufficiently strained to show the characteristic behavior of the plates. Fig. 17*e* shows the beginning of a fracture around one of the metallic globule inclusions before the surrounding metal has been very severely strained. For this reason the influence of the plates on the mechanical properties of the crystals can not be stated with certainty. It would appear, however, that on account of the apparently unavoidable unsoundness of the metal, any possible influence of the nitride plates upon the mechanical properties of the material is quite negligible.

Some of the same specimens used for cold bending were torn partially in two after localizing the tear by means of a saw-cut in the edge of the plate. The specimen was then copper plated and prepared for microscopic examination, the surface having been ground away sufficiently to reveal the weld metal with the tear in it. Fig. 18 shows the appearance of the extreme end of a tear produced in this manner. The nitride plates do not appear to

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have determined to any extent the path taken in the rupture produced in this manner.

3. EFFECT OF HEAT TREATMENT UPON STRUCTURE

With the view of possibly gaining further information as to the nature of the plates (assumed to be nitride) which constitute such a characteristic feature of the microstructure, a series of heat treatments was carried out upon several specimens of arc-fused electrodes of both types. Briefly stated, the treatment consisted in quenching the specimens in cold water after heating them for a period of 10 or 15 minutes at a temperature considerably above that of the Ac_3 transformation; 925°, 950°, and 1000° C were the temperatures used. After microscopical examination of the different quenched specimens they were tempered at different temperatures, which varied from 650 to 925° C for periods of 10 to 20 minutes. The samples which were used were rather small in size, being only one-eighth of an inch thick in order that the effect of the treatment should be very thorough,

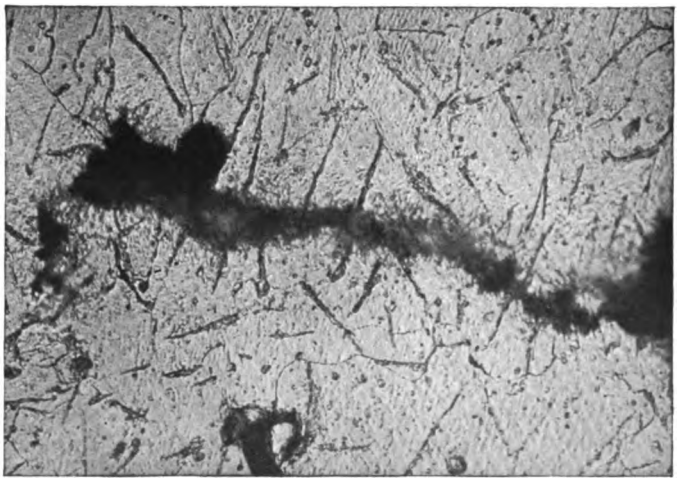

FIG. 18.—*Microstructure of arc-fused steel, showing the relation of the path of rupture to the "nitride plates."* × 500

A thin plate of metal (specimen of Fig. 17) was slowly torn after being partially sawed in two. The tear bears no evident relationship to the plates that lie in its path. Etching reagent used before fracturing the metal, 2 per cent alcoholic solution of nitric acid

and were taken from test bars A–2, A–6, AD–10, B–2, B–6, and B–9. These represented metal which had been deposited under different conditions of current density as shown in Table 6. No plates were found to be present in any of the specimens after quenching. Fig. 19 shows the appearance of one of the quenched

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bars, which condition is typical of all. The structure indicates that the material comprising the plates had dissolved in the matrix of iron and had been retained in this condition upon quenching. The needlelike striations within the individual grains are characteristic of the condition resulting from the severe quenching and are to be observed at times in steel of a very low carbon content. Fig. 20 shows the appearance of one of the A electrodes (five thirty-seconds of an inch) quenched in cold water from 1000° C. Some of the crystals of the quenched iron also show interior markings somewhat similar in appearance to the nitride plates (Fig. 20*b*). These are, however, probably of the same nature as the interior treelike network sometimes seen in ferrite which has been heated to a high temperature.[6]

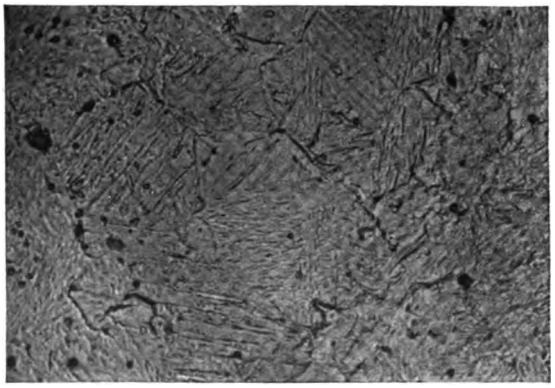

FIG. 19.—*Microstructure of arc-fused steel after quenching in water from 1000° C.* × 500

The plates characteristic of the material (specimen AD-10) before heating have disappeared and a martensitic pattern has been produced. Etching reagent, 5 per cent alcoholic solution of picric acid

The striations were found to be most pronounced in the specimens of arc-fused metal which were quenched from the highest temperatures, as might be expected. Braune [49] states that nitride of iron in quenched metal is retained in solution in the martensite. The same may be inferred from the statement by Giesen [43] "that in hardened steel, it (nitrogen) occurs in martensite." Ruder [28] has also shown that nitrogenized electrolytic iron (three hours at 700° C in ammonia), after being water quenched from a temperature of 600 to 950° C, shows none of the plates which were present before the specimen was heated.

[6] H. S. Rawdon and H. Scott, B. S. Sci. Papers, No. 356, Fig. 2*b*.

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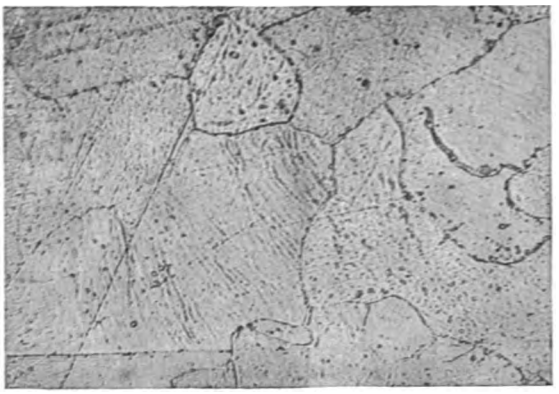
The sets of specimens (A–2, A–6, AD–10, B–2, B–6, and B–9) quenched from above the temperature of the Ac_3 transformation were heated to various temperatures, 650, 700, 800, and 925° C.



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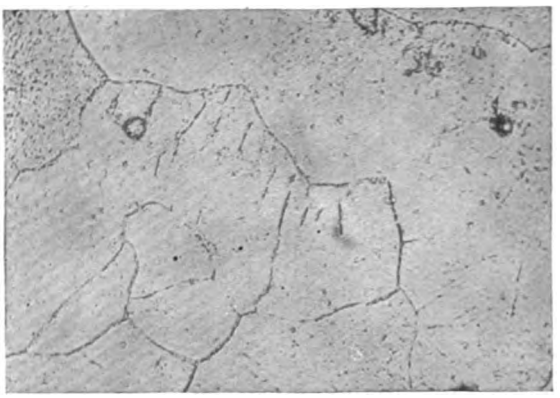

B

FIG. 20.—*Microstructure of "carbonless" steel after quenching in water from 1000° C.* × 500

A sample of electrode metal, type A, $\frac{1}{4}$ inch diameter, was used, being maintained 10 minutes at temperature before quenching. *A*, Some of the ferrite crystals (particularly near the surface of the specimen) show a faint pattern similar to that of Fig. 19; *B*, other ferrite crystals show interior markings somewhat suggestive of the plates of the arc-fused metal. Etching reagent, 2 per cent alcoholic solution of nitric acid

In all cases the specimens were maintained at the maximum temperature for approximately 10 to 15 minutes and then cooled in the furnace. Figs. 21 to 24, inclusive, summarize the resulting effects upon the structure. Heating to 650° C is not sufficient to allow the plates to redevelop, but in the specimens heated to 700° C a few small ones were found. The effect is progressively more pronounced with the increased temperature of tempering,

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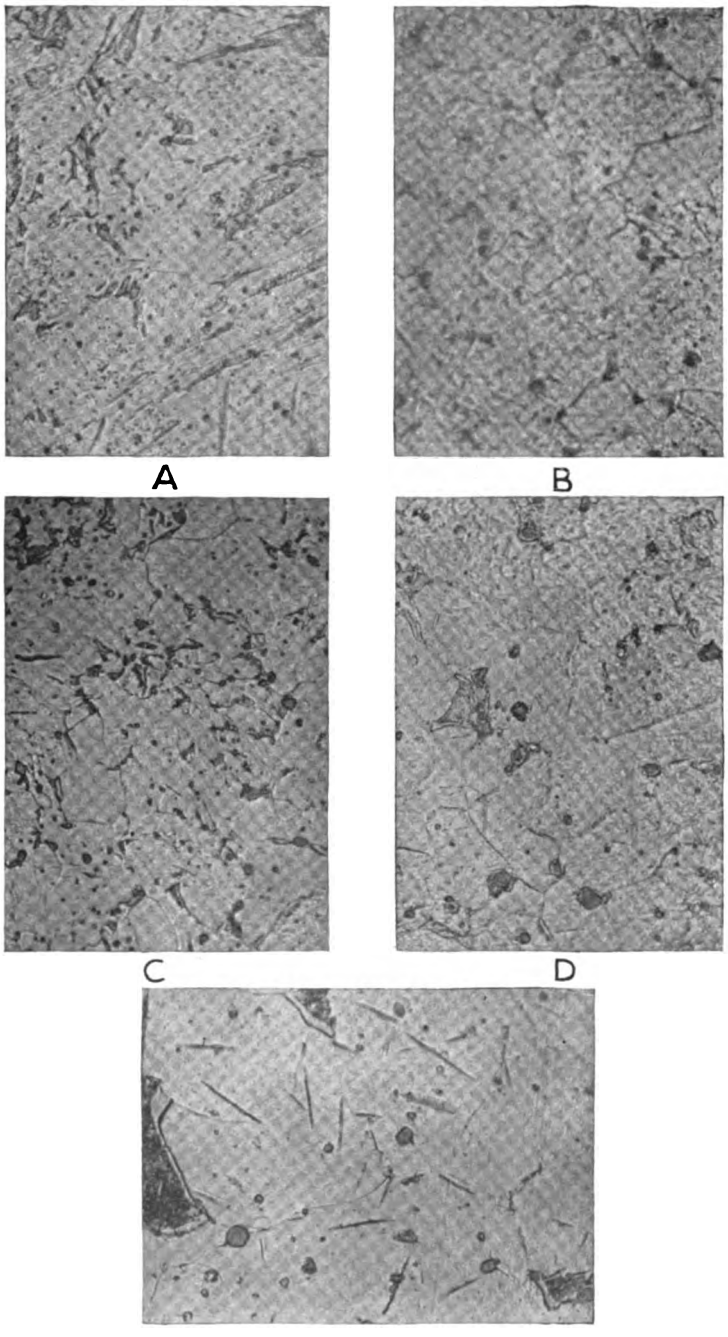

FIG. 21.—*Microstructure of arc-fused steel, Type A, showing the effect of heat treatment.* × 500

The specimen A-2 (Table 6) was quenched in water from approximately 1000° C (considerably above Ac_3 temperature, Fig. 27), and reheated as shown below. *A*, arc-fused metal as deposited; *B*, same as *A* after quenching and subsequently reheating to 650° C; no plates have formed during this treatment: *C*, a quenched specimen was reheated to 700° C; a few plates have appeared; *D*, a quenched specimen was reheated to 800° C; *E*, a quenched specimen was reheated to 925° C. Etching reagent, 2 per cent alcoholic solution of nitric acid

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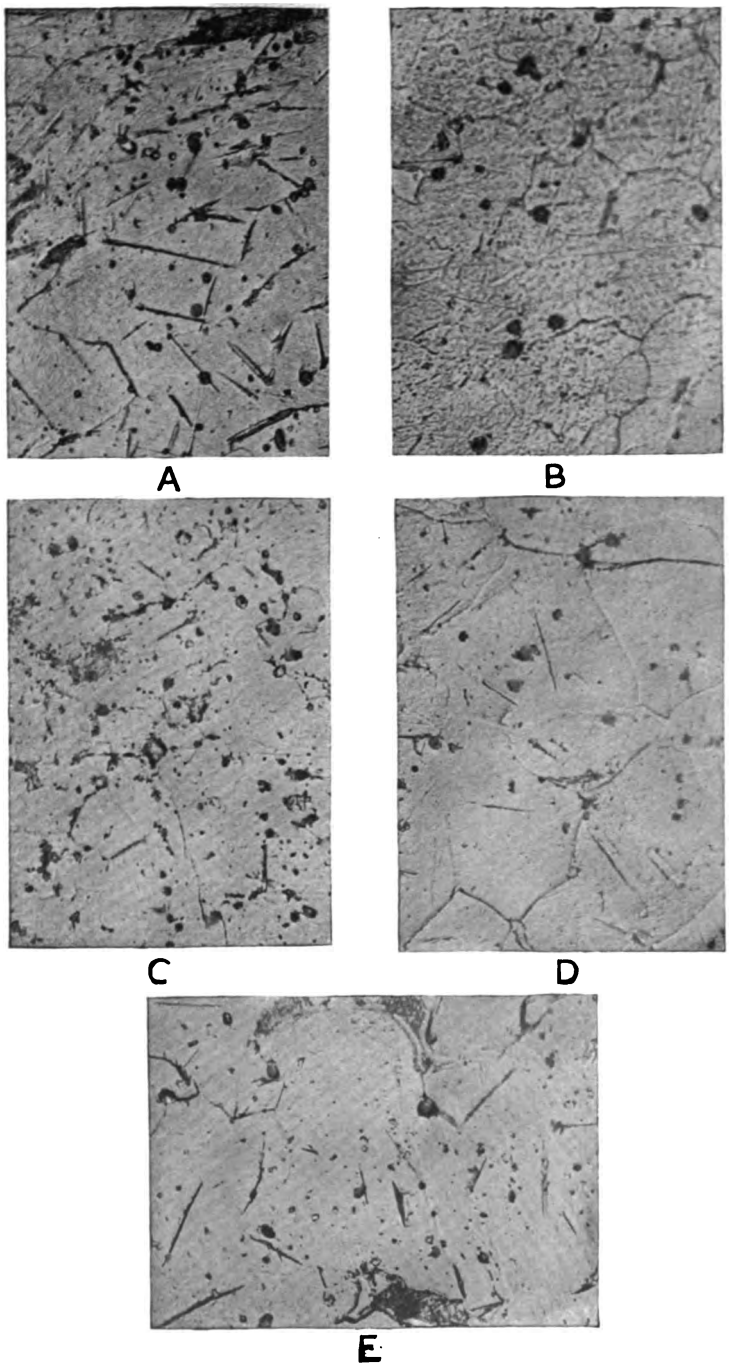

FIG. 22.—*Microstructure of arc-fused steel, type A, showing the effect of heat treatment.* × 500

The specimen AD–10 (Table 6) was quenched in water from approximately 1000° (C considerably above Ac_3 temperature, Fig. 27) and reheated as shown below. *A*, arc-fused metal as deposited; *B*, same as *A* after quenching and subsequently reheating to 650° C; no "plates" have been formed by this treatment; *C*, a quenched specimen was reheated to 700° C; a few plates have appeared; *D*, a quenched specimen was reheated to 800° C; *E*, a quenched specimen was reheated to 925° C. Etching reagent, 2 per cent alcoholic solution of nitric acid

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and in the material heated to 925° C they are as large and as numerous as in any of the arc-fused specimens. The heating also develops the islands of pearlite, which are not always to be distinguished very clearly in the simple fused metal. The work of Ruder [**28**], referred to above, shows that nitrogenized iron which has been quenched and so rendered free from the nitride plates behaves in a similar manner upon heating to temperatures varying from 700 to 950° C; the plates reappear after a heating for 15 minutes at 700° C (or above), followed by a slow cooling. The similarity in behavior of the two is a further line of evidence that the arc-fused metal contains more or less nitrogenized iron throughout its mass.

The persistence of the nitride plates was also studied in specimens heated at 1000° C in vacuo for periods of six and of four and one-half hours. A set of specimens (one each of test bars A–D2, A–3, AD–6, A–10, B–2, B–4, B–5, and BD–5) was packed in a Usalite crucible, and covered with alundum "sand"; this crucible was surrounded by a protecting alundum tube and the whole heated in an Arsem furnace. A vacuum equivalent to 0.2 mm mercury was maintained for the greater part of the six-hour heating period; for the remainder of the time the vacuum was equivalent to 1.0 to 0.2 mm mercury. The specimens were allowed to cool in the furnace. Ruder [**24**] has stated that one hour's heating in vacuo at 1000° C was sufficient to cause a marked diminution in the number of plates in both arc-weld material and nitrogenized iron, and that at 1200° C they disappeared entirely.

The results obtained are shown in Fig. 25. In contradistinction to Ruder's work, the plates are more conspicuous and larger than before, and the oxide specks are larger and fewer in number. It would appear that the conditions of the experiment are favorable for a migration of the oxide through an appreciable distance and for a coalescing into larger masses. The oxide is eliminated entirely in a surface layer averaging approximately 0.15 mm in depth. Only in projections (right-angled corners, sections of threads of the tension bar, etc.) was there any removal of the nitride plates by the action of the continued heating in vacuo. This is shown in Fig. 26*c*, which illustrates the removal of the oxide inclusions also. No evidence was found that the small amount of carbon present in the arc-fused metal is eliminated, particularly very far beneath the surface.

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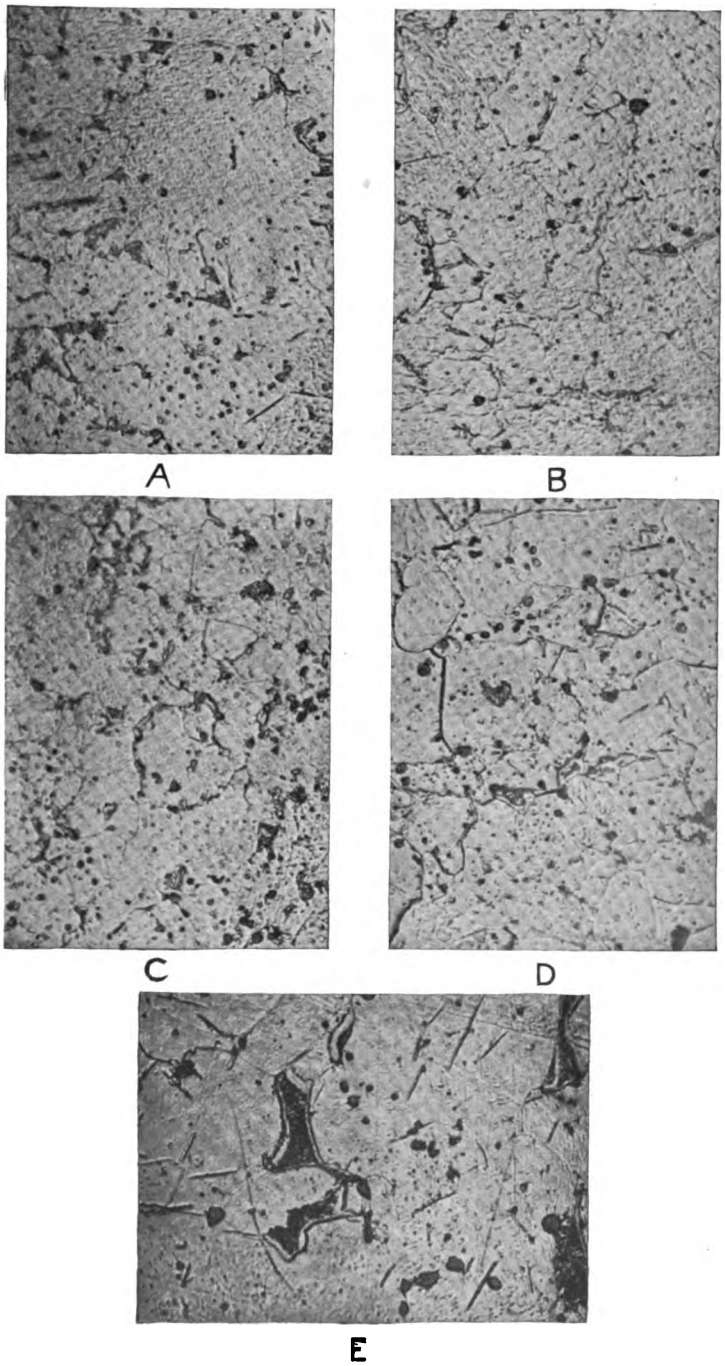

FIG. 23.—*Microstructure of arc-fused steel, type B, showing the effect of heat treatment.* × 500

The specimen B-2 (Table 6) was quenched in water from approximately 1000° C and reheated as shown below. *A*, arc-fused metal as deposited; *B*, a quenched specimen was reheated to 650° C; no plates have appeared after this treatment; *C*, a quenched specimen was reheated to 700° C; a few plates have appeared; *D*, a quenched specimen was reheated to 800° C; *E*, a quenched specimen was reheated to 925° C. Etching reagent, 2 per cent alcoholic solution of nitric acid

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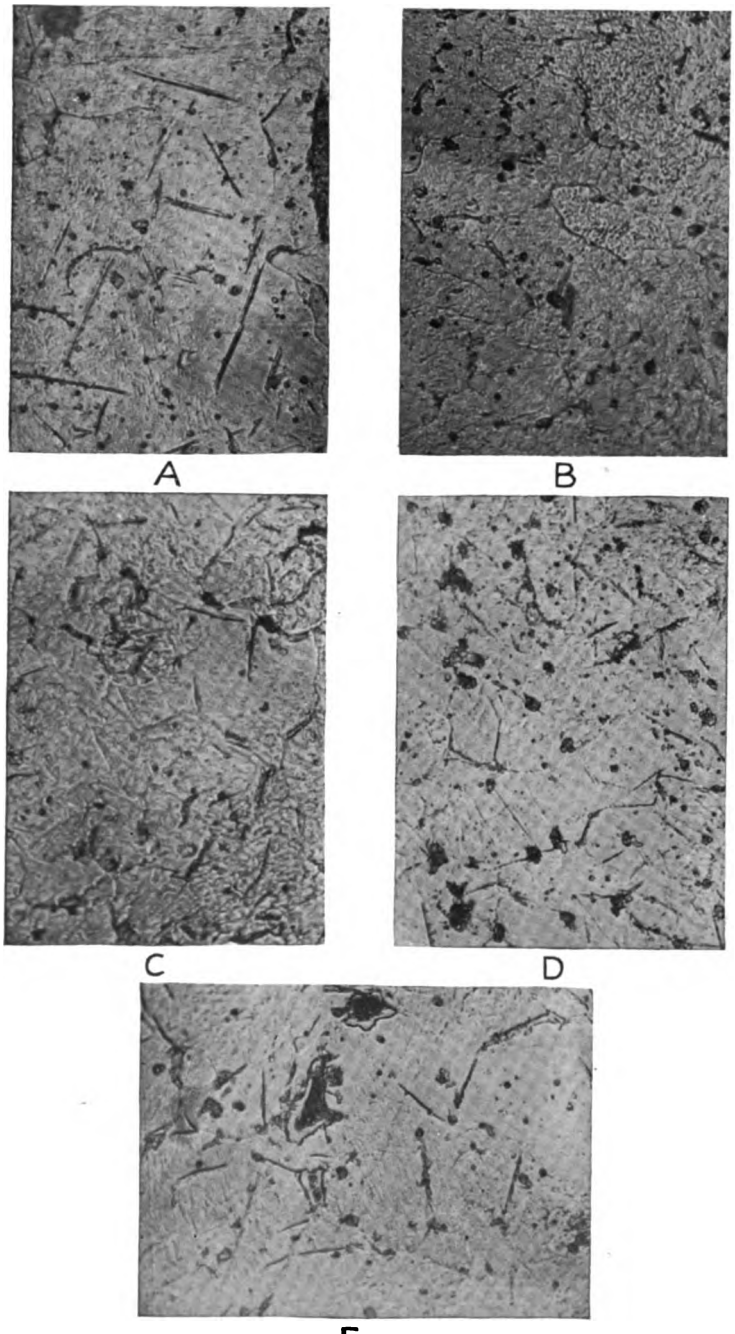

FIG. 24.—*Microstructure of arc-fused steel, type B, showing the effect of heat treatment.* × 500

The specimen B–9 (Table 6) was quenched in water from approximately 1000° C and reheated as shown below. *A*, arc-fused metal as deposited; *B*, a quenched specimen was reheated to 650° C; no plates were formed by this treatment; *C*, a quenched specimen was reheated to 700° C; a few plates have appeared; *D*, a quenched specimen was reheated to 925° C. Etching reagent, 2 per cent alcoholic solution of nitric acid

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Fig. 12*c* illustrates an interesting exception to the rule that the nitride plates are flat. In the metallic globule inclusion shown the plates have a very pronounced curve. The general appearance suggests that the "metallic globules" solidified under a condition of constraint, and that this condition still persists even after the six hours' heating at 1000° C which the specimen received.

Several of the specimens which were heated in vacuo were analyzed for nitrogen. The results are given in the table below.

TABLE 9.—Change in Nitrogen Content of Arc-Fused Metal upon Heating[a]

Specimen	Weight of sample	Hours of heating in vacuo at 1000° C	Average nitrogen content		Loss
			Before heating	After heating	
	Grams		Per cent	Per cent	Per cent
A-3	1.39	6	0.127	0.062	51
B-4	6.0	6	.124	.078	37
BD-5	1.62	6	.140	.059	57
B-5	1.16	6	.121	.054	55
AD-2	1.28	10½	.149	.043	71
B-2	1.76	10½	.152	.050	67
AD-6	2.23	10½	.134	.061	54

[a] The method used for the determination of the nitrogen gives only the *nitride* nitrogen; hence a possible explanation for the change in the nitrogen content is that it has been converted into another form than nitride and may not have been eliminated as the results apparently indicate.

The fact that the specimens lose nitrogen upon heating, although the amount remaining is still many times the nitrogen content of the metal before fusion, coupled with the fact that the "nitride plates" are larger and more conspicuous after heating than before, suggests very strongly that these plates are not simple nitride of iron. In some of the specimens heated in vacuo at 1000° C for the longer period many of the crystals contain the smaller or secondary lines (Fig. 26*a*), previously referred to (p. 33). This would indicate that these secondary lines represent a transition stage. In some of the crystals (Fig. 26*b*) the lines have assumed an appearance which suggests that they are a crystalline phenomenon somewhat similar to Neumann bands.

4. THERMAL ANALYSIS OF ARC-FUSED STEEL[7]

In order to throw further light on the nature of the plates (nitride) found in the metal after fusion in the arc, the thermal characteristics of the electrode material before and after fusion as revealed by heating and cooling curves were determined.

[7] Credit is due to Howard Scott, assistant physicist, for the work described in this section.

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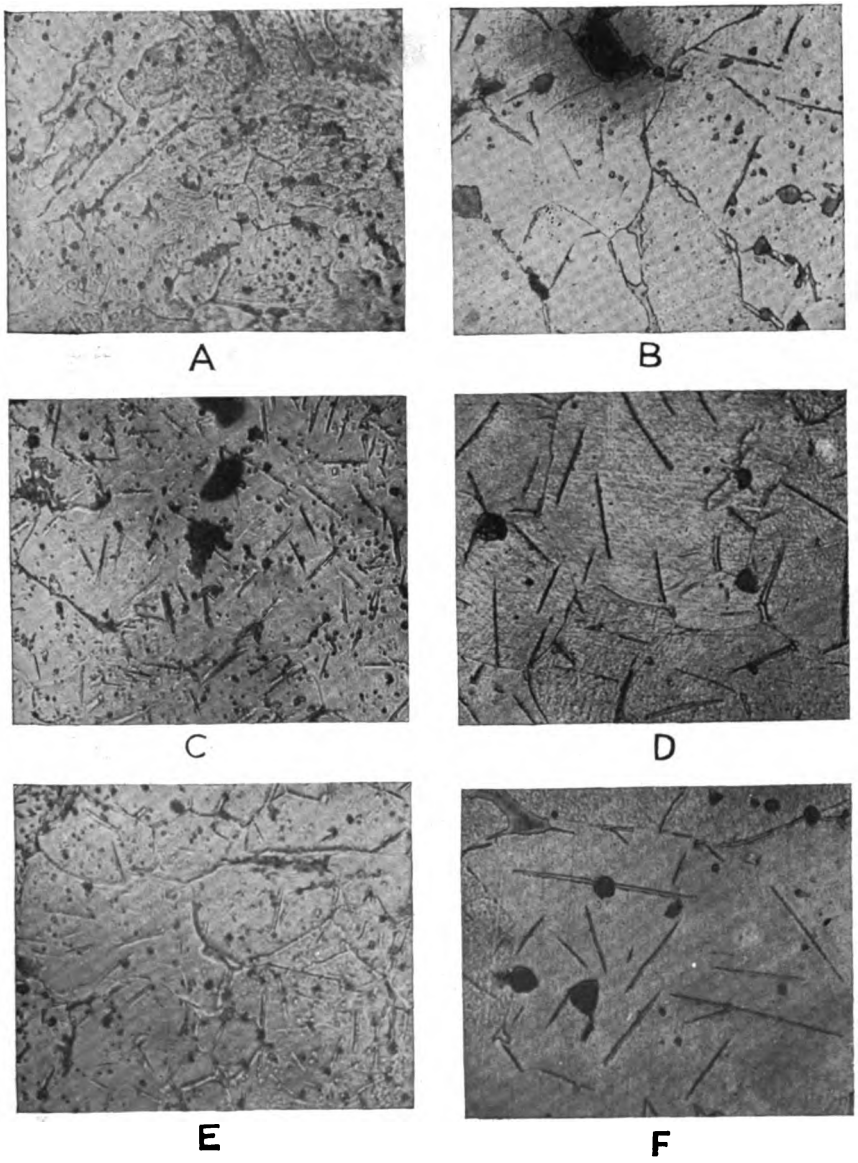

FIG. 25.—*Microstructure of arc-fused iron showing the effect of heating in vacuo at a high temperature.* × 500

The specimens were heated continuously for 6 hours at 1000° C in vacuo. *A*, specimen AD–2, initial structure; *B*, same as *A*, after heating; *C*, specimen B–4, initial structure; *D*, same as *C* after heating; *E*, specimen A–10, initial structure; *F*, same as *E*, after heating. *B*, *D*, and *F* all show some cementite at grain boundaries which resulted from the "divorcing" of pearlite. Many of the plates appear to have been influenced in their position by an oxide globule. Etching reagent, 2 per cent alcoholic solution of nitric acid

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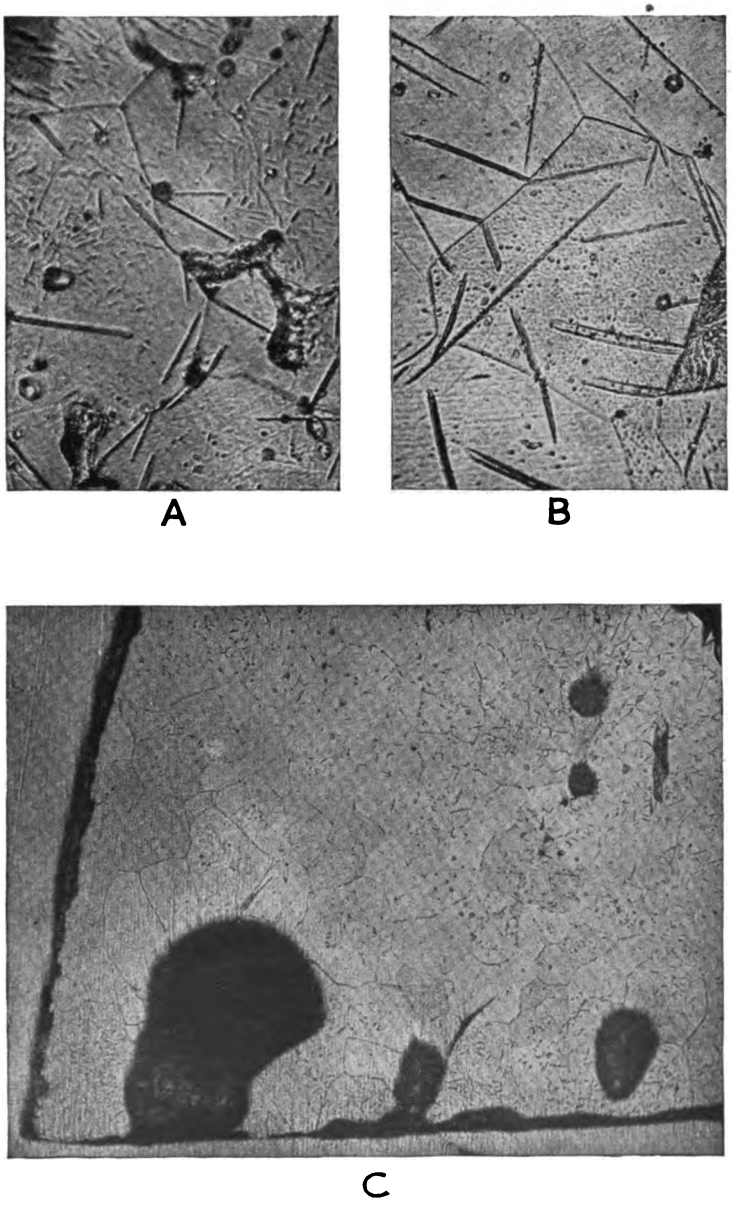

FIG. 26.—*Microstructure of arc-fused steel showing the effect of prolonged heating at high temperature*

A, specimen AD–2 (Table 6) was heated in vacuo for 10½ hours at 1000° C in stages of 6 and 4½ hours; two generations of "plates" have appeared after this treatment, × 500; *B*, specimen AD–6 (Table 6) heated as in *A* above; the plates which have persisted are relatively large and resemble Neumann bands in appearance, × 500; *C*, specimen AD–10 (Table 6) heated 6 hours in vacuo at 1000° C; the micrograph represents a corner of the specimen; the oxide and the "nitride plates" have been removed at the exposed corner, × 100

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Samples of a three-sixteenths-inch electrode of type A and of the specimen, A–5, which resulted from the fusion were used as material (composition in Tables 3 and 6).

In Fig. 27 are given the curves obtained which show the characteristic behavior of the arc-fused metal upon heating. The commonly used inverse-rate method was employed in plotting the data; the details of manipulation and the precautions necessary for the thermal analysis have already been described.[8] In Table 10 are summarized the data shown graphically in Fig. 27.

TABLE 10.—The Thermal Characteristics of Arc-Fused Iron

Rate of heating and cooling	Ac_3 maximum	Ac_3				Time above A_3	Ar_3			Ar_3 maximum
		Beginning	Maximum	End	Maximum temperature		Beginning	Maximum	End	
°C sec.	°C	°C	°C	°C	°C	Minutes	°C	°C	°C	°C
[a] 0.16	768	892	910	918	960	.....	896	893	879	766
[a] .15										
.15	765	897	911	916	960	.....	895	891	879	766
UNFUSED ELECTRODE										
ARC-FUSED METAL [b]										
0.14	764	.....	847	874	960	28	847	838	820	764
.13	764	.....	849	876	985	42	847	836	822	764
.13	764	.....	844	870	960	29	847	837	821	765
.13	766	.....	850	874	1035	256	848	835	816	764

[a] Heated at rate of 0.16° C per second, cooled at 0.15° C per second; for other specimens the rate of cooling equaled the rate of heating.

[b] The same specimen was heated four times in succession, as shown.

The principal change to be noted which has resulted from the arc fusion of the iron is in the A_3 transformation. This is now very similar to the corresponding change observed in a very mild steel (for example, approximately 0.15 per cent carbon). That the difference in the A_3 transformation of the arc-fused metal as compared with that of the original electrode is not due to an increase in the carbon content is evident from the lack of the sharp inflection of the A_1 transformation ("pearlite point"), which would, of necessity, be found in a low-carbon steel. No evidence of the A_1 change was observed for the arc-fused iron within the range of temperature, 150 to 950° C. The change in the character of the A_3 transformation is without doubt to be attributed to the influence of the increased nitrogen content of the iron.

[8] H. Scott and J. R. Freeman, B. S. Sci. Papers, No. 348.

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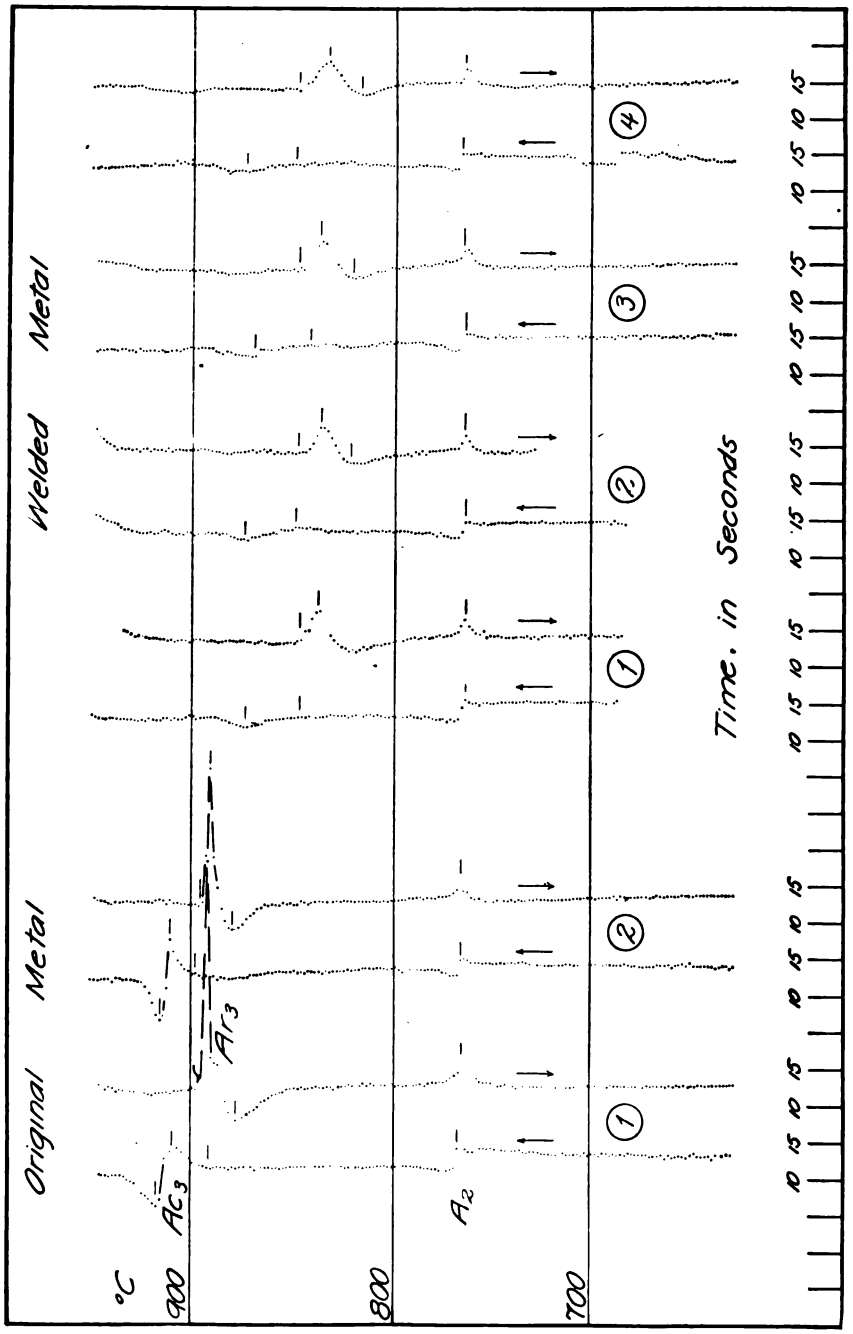

FIG. 27.—*Thermal characteristics of arc-fused steel*

The direction of arrow indicates whether the curve is a heating or a cooling curve. The numerals indicate the number of "runs" taken on the same specimen

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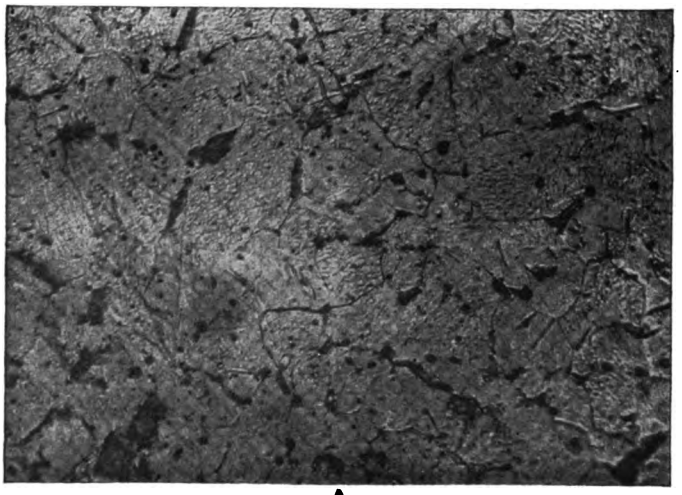
The specimen was maintained above the temperature of the A_3 transformation for a total period (four heatings) of six hours, the



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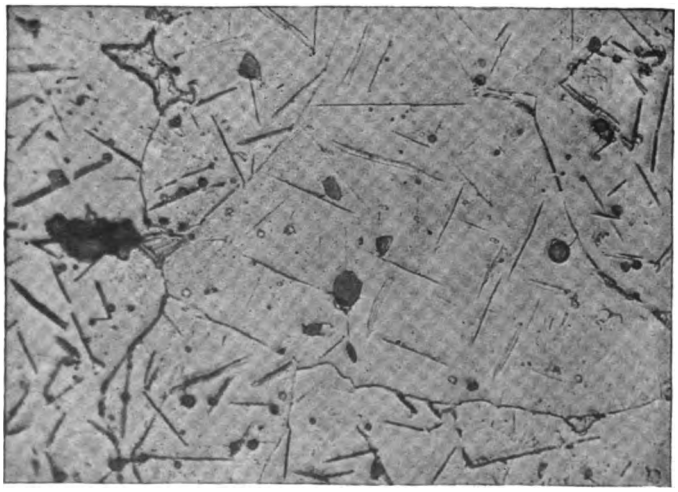

B

FIG. 28.—*Microstructure of the arc-fused metal used in thermal analysis.* × 500

The specimen A–5 was heated four times as shown in Fig. 27. The total length of time above the temperature of the Ac_3 transformation was 6 hours. *A*, initial structure as fused; *B*, structure after heating. Etching reagent, 2 per cent alcoholic solution of nitric acid

maximum temperature being 1035° C. The transformation apparently is unaffected by the long-continued heating, thus confirming the results described in the preceding section. In

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Fig. 28 are given micrographs to show the change of structure occurring in the specimen.

In discussing the properties of steel nitrogenized by melting it in nitrogen under pressure, Andrew [38] states that it was found possible to extract almost entirely the small quantities of nitrogen by heating a specimen at 1000° C in vacuo for periods of one to six hours. The metal used contained 0.16 per cent carbon and 0.3 per cent nitrogen. Thermal curves are given to show that there are no critical transformations in the material; the nitrogen suppresses them. They gradually reappear, however, as the nitrogen is removed by heating the material in vacuo at 1000° C. Several days' heating was required, however, to obtain an entirely degasified product, the carbon also being removed. A further statement is made that a steel of 0.6 per cent carbon content containing 0.25 per cent nitrogen can be brought back to the normal state of a pure steel only by several weeks' heating in vacuo.

The results of the thermal analysis add considerable confirmatory evidence to support the view that the plates existing in the arc-fused metal are due to the nitrogen rather than to carbon.

VIII. DISCUSSION OF RESULTS

In any consideration of electric-arc welding it should be constantly borne in mind that the weld metal is simply metal which has been melted and has then solidified in situ. The weld is essentially a casting, though the conditions for its production are very different from those ordinarily employed in the making of steel castings. The metal loses many of the properties it possesses when in the wrought form, and hence it is not to be expected that a fusion weld made by any process whatever will have all the properties that metal of the same composition would have when in the forged or rolled condition. A knowledge of the characteristic properties of the arc-fused iron is then of fundamental importance in the study of the electric-arc weld.

The peculiar conditions under which the fusion took place also render the metal of the weld quite different from similar metal melted and cast in the usual manner. It is seemingly impossible to fuse the metal without serious imperfections. The mechanical properties of the metal are dependent, therefore, to an astonishing degree upon the skill, care, and patience of the welding operator. The very low ductility shown by specimens when stressed in tension is the most striking feature observed in the mechanical

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properties of the material as revealed by the tension test. As explained above, the measured elongation of the tension specimen does not truly indicate a property of the metal. Due to the unsoundness, already described in the discussion of the structure, the true properties of the metal are not revealed by the tension test to any extent. The test measures, largely for each particular specimen, the adhesion between the successively added layers, which value varies considerably in different specimens on account of the unsoundness caused by imperfect fusion, oxide, and other inclusions, tiny inclosed cavities, and similar undesirable features. The elongation measured for any particular specimen is due largely, if not entirely, to the increase of length due to the combined effect of the numerous tiny imperfections which exist throughout the sample.

That the metal is inherently ductile, however, is shown by the behavior upon bending as recorded in the microstructure of the bent specimen. The formation of slip bands within the ferrite grains to the extent which was observed is evidence of a high degree of ductility. It appears, however, that the grosser imperfections are sufficient to prevent any accurate measurement of the real mechanical properties of the metal from being made. The conclusion appears to be warranted, therefore, that the changes of composition which the fusion entails, together with the unusual features of microstructure which accompany the composition change, are of minor importance in determining the strength, durability, and other properties of the arc weld.

The view that the characteristic features observed in the structure of the arc-fused iron are due to the increased nitrogen content is supported by several different lines of evidence. These include the likeness of the structure of the material to that of pure iron which has been "nitrogenized," the similarity in the behavior of both arc-fused and nitrogenized iron upon heating, the evidence shown by thermal analysis of the arc-fused metal, together with the fact that, as shown by chemical analysis, the nitrogen content increases during fusion while the other elements, aside from oxygen, decrease in amount. The characteristic form in which oxide occurs in iron, together with its behavior upon heating, renders very improbable the assumption that the oxide is responsible for the plates observed in the material.

In arc-fusion welds in general, the mass of weld metal is in intimate contact with the parts which are being welded, so that it is claimed by many that because of the diffusion and inter-

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mingling of the metal under repair with that of the weld the properties of the latter are considerably improved. The comparison shown in the table below somewhat supports this claim. The nearest comparison found available with this Bureau's specimens are some of those of the welds designated as the "Wirt-Jones" series reported by H. M. Hobart [1]. These welds were of the 45° double-V type made in one-half-inch ship plate; the specimens for test were of uniform cross section 1 by one-half inch, the projecting metal at the joint having been planed off even with the surface of the plates and the test bars were so taken that the weld extended transversely across the specimen near the center of its length. The electrodes used were similar to those designated as type B in this Bureau's investigation.

TABLE 11.—Comparison of Mechanical Properties of Welds with Results of Tests of Arc-Fused Metal Prepared under Similar Conditions

Bureau of Standards				Wirt-Jones			
Diameters of electrodes	Direct current	Ultimate tensile strength	Elongation in 2 inches	Diameter of electrode[a]	Direct current	Ultimate tensile strength	Elongation in 2 inches
	Amp.	Lbs./in.²	Per cent		Amp.	Lbs./in.²	Per cent
1/8 inch [a]	110	52 650	7.5	1/8 inch	110	45 800	8.0
1/8 inch [b]	110	49 050	9.0	1/8 inch	115	58 200	14.0
1/8 inch [b]	110	44 400	6.5	1/8 inch	115	59 400	13.5
				1/8 inch	120	53 700	7.0
				1/8 inch	120	57 600	8.5
Average		48 700	7.7	Average		54 940	10.2
5/32 inch [a]	145	46 450	5.0	11/64 inch	150	60 900	8.0
5/32 inch [b]	145	48 130	8.0	5/32 inch	155	62 600	11.5
5/32 inch [b]	145	41 750					
Average		45 440	6.3	Average		61 750	9.8
5/32 inch [a]	185	49 600	7.5	11/64 inch	175	59 800	9.0
5/32 inch [b]	185	49 086	12.5				
5/32 inch [b]	185	47 100	11.0				
Average		48 595	10.3				

[a] Electrodes were used in bare condition. [b] Electrodes were coated as previously described.

Since the specimens used in work described in the foregoing sections were prepared in a manner quite different from the usual practice of arc welding, no definite recommendations applicable to the latter can be made. It appears, however, from the results obtained that the two types of electrodes used—that is, "pure" iron and low-carbon steel—should give very similar results in practical welding. This is due to the changes which

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occur during the melting, so that the resulting fusions are essentially of the same composition. The use of a slight coating on the electrodes does not appear to be of any material advantage so far as the properties of the resulting fused metal are concerned. Since the program of work as carried out did not include the use of any of the covered electrodes which are highly recommended by many for use in arc welding, particularly for "overhead work," no data are available as to the effect of such coatings upon the properties of the metal resulting from fusion. Although all of the specimens used in the examinations were made by the use of direct current, it appears from the results obtained with a considerable number of welds representing the use of both kinds of current, submitted for the preliminary examinations, that the properties of the fused metal are independent of the kind of current and are influenced primarily by the heat of fusion. Any difference in results obtained by welding with alternating current as compared with those obtained with direct current apparently depends upon the relative ease of manipulation during welding rather than upon any intrinsic effect of the current upon properties of the metal.

IX. SUMMARY

1. A fusion weld is fundamentally different from all other types in that the metal of the weld is essentially a casting. The arc-fusion weld has characteristics quite different from other fusion welds. A preliminary study of a considerable number of specimens welded under different conditions confirmed this general opinion concerning arc welds. A knowledge of the properties of the arc-fused metal is fundamental in any study of electric-arc welding.

2. Blocks of arc-fused metal of size large enough to permit a tension specimen (0.505 inch diameter, 2-inch gage length) to be taken from each were made under different conditions of fusion. Two types of electrodes, a "pure" iron and a mild steel, were used in the bare condition and also after receiving a slight coating. With these were included a set of similar specimens prepared outside of this Bureau by expert welding operators.

3. During the fusion the composition of the metal of both types of electrodes is changed considerably by the burning-out of the carbon and other elements, and the two become very much alike in composition. A very considerable increase in the nitrogen content occurs at the same time.

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4. The mechanical properties of the arc-fused metal as measured by the tension test are essentially those of an inferior casting. The most striking feature is the low ductility of the metal.

5. All of the specimens showed evidence of unsoundness in their structure, tiny inclosed cavities, oxide inclusions, lack of intimate union, etc. These features of unsoundness are, seemingly, a necessary consequence of the method of fusion as now practiced. They determine almost entirely the mechanical properties of the arc-fused metal. The observed elongation of the specimen under tension is due to the combined action of the numerous unsound spots rather than to the ductility of the metal. That the metal is inherently ductile, however, is shown by the changes in the microstructure produced by cold bending. By taking extreme precautions during the fusion, a great deal of the unsoundness may be avoided and the mechanical properties of the metal may be considerably improved. However, the specimens as described are more representative of actual present practice in welding.

6. A very characteristic feature of the microstructure of the arc-fused metal is the presence of numerous plates within the ferrite crystals, arranged usually along crystallographic planes. These persist in the metal after prolonged heating; for example, ten hours at 1000° C in vacuo does not remove them. The various lines of evidence available indicate that they are related to the nitrogen content of the metal, which is increased during the fusion.

7. Upon heating the metal above the temperature of the Ac_3 transformation, the nitride enters into solid solution in the ferrite and is retained so by quenching. Upon heating this quenched metal, the plates reappear and begin to show at a temperature of approximately 700° C. The effect of nitrogen upon the Ar_3 transformation is very similar to that of an equal amount of carbon. However, there is no evidence of a transformation corresponding to the A_1 or the pearlite change due to the nitrogen in the steel.

8. The microscopic examination indicates that there is but little, if any, relation between these nitride plates and the path of rupture produced by tension. The effect of grosser imperfections of the metal is so much greater than any possible effect of the nitride plates in determining the mechanical properties that the conclusion appears to be warranted that this feature of the structure is a matter of relatively minor importance in ordinary arc welds.

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9. Judged from the results obtained, neither type of electrode appears to have a marked advantage over the other. The use of a slight protective coating on the electrodes does not appear to affect materially in any way the mechanical properties of the arc-fused metal.

10. The specimens were prepared in a manner quite different from that used ordinarily in electric-arc welding, and the results do not justify any specific recommendations concerning methods of practice in welding.

Credit is due to H. L. Whittemore and R. W. Woodward for aid in the necessary mechanical testing, and to F. H. Tucker for the general chemical analyses made.

WASHINGTON, June 7, 1920.

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-Note-

Doubt has arisen from the results of some special etching tests, described by C. F. Comstock in his discussion of "Nitrogen in Steel, and the Erosion of Guns", H. E Wheeler, Trans. Amer. Inst. of Mining and Met. Engrs., 1920, and tried out by the authors several months following the preparation of this paper, as to the correctness of the identification of the microconstituent designated as "pearlite islands" (See Figs. 21e and 24e), The strong similarity in appearance of these "pearlite islands" to the pearlite in ordinary low-carbon steels, as developed by the usual etching reagents, picric acid, nitric acid, etc., and apparent compatability of the carbon content of the arc-fused iron with the extent of the "pearlite islands" noted, naturally led to the assumption that this constituent was pearlite, particularly as there was found in many of the "pearlite islands" in the slowly-cooled samples a border that extended part way around the edge very similar to cementite atolls in partially divorced pearlite. It had been noted, however, that these "pearlite islands" possessed in many cases a rather more angular-shaped appearance than is generally found.

The results of the above-mentioned tests failed to prove these "pearlite islands" to be really pearlite as usually formed in carbon steels. It might appear from the similarity in behavior during etching to that of the needles or plates (assumed to be iron nitride), that this constituent owes its existence to the prese nce of nitrogen, though there is no clear proof for this. Whether the small amount of carbon, as indicated by chemical analysis to be present in the metal, had a contributory influence upon the formation of this constituent or not is not clear. Wilson (Chem. & Met. Chem. 23, p. 1161, 1920) definitely states that this pearlite-like constituent is an iron-iron nitride eutectoid though no substantiating evidence is given.

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CAUSES AND PREVENTION OF THE FORMATION OF NONCONDENSIBLE GASES IN AMMONIA ABSORPTION REFRIGERATION MACHINES

BY

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Bureau of Standards

OCTOBER 25, 1920

PRICE, 5 CENTS

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CAUSES AND PREVENTION OF THE FORMATION OF NONCONDENSIBLE GASES IN AMMONIA ABSORPTION REFRIGERATION MACHINES

By E. C. McKelvy and Aaron Isaacs [1]

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1. INTRODUCTION

One of the great difficulties encountered in the operation of an ammonia absorption refrigeration machine is the formation of foul or noncondensing gases. The presence of these gases is usually indicated by the increase in the operating pressure of the ammonia plant. If a sample of liquor from the generator of such a plant be taken, it will always be colored a deep brown and will deposit a sediment upon standing, thereby showing that corrosion has accompanied the gas formation within the generator. However, the corrosion of the interior of the generator is of secondary importance. The foul gas causes the most trouble, since the plant must be purged of this gas just as often as it forms, with resulting loss of ammonia. The presence of foul gas makes it necessary to operate the plant at higher temperatures and pressures, thereby causing greater leakage, more corrosion, more danger from possible explosions, and increased wear on the plant. The noncondensing nature of the gas causes decreased efficiency of the compressor and consequently decreased efficiency of operation of the refrigeration plant as a whole.

In order to study the composition of the gases formed, samples from a number of plants were collected and analyzed. The per-

[1] This investigation was completed prior to the death in November, 1919, of the senior author, but the manuscript was prepared subsequently by the junior author.

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centage composition of the different samples varied considerably, but in every case the gas consisted of nitrogen, hydrogen, and oxygen. In some cases the hydrogen content was as high as 98 per cent, while in others it was as low as 1 or 2 per cent. Where the hydrogen content was very small in amount the nitrogen content was very large, and vice versa. The oxygen was usually only a minor constituent and was never present in any appreciable quantity unless the nitrogen content was very high. It was soon recognized that the nitrogen and hydrogen must come from different sources and could not come from the decomposition of the ammonia, even though in many cases the gases were present in approximately the proportions to form ammonia. Ammonia will not decompose into its constituents under the conditions of operation of an absorption plant. However, there was a possibility of the formation of nitrides or amines. Some of the sludge from the absorption plant at the Bureau of Standards, which was giving much trouble at that time, was tested for these by dissolving in acid, making the solution alkaline and distilling into standard acid. Under this treatment ammonia would be produced if any amines or nitrides were present. There was, however, no evidence of the presence of either of these compounds. This fact pointed strongly toward the conclusion that in those plants where nitrogen, or a mixture of nitrogen and oxygen, was found in the gas, these impurities were caused by air that had leaked into the system. The action of air within such a system, therefore, was one of the points that were studied. The presence of hydrogen had to be explained in another manner, and its presence in such large quantities as were found in some plants could not be accounted for on the basis of the interaction of either pure water or pure ammonia with the iron. Therefore the effect of possible impurities in ammonia upon its corrosive action was studied in order to find out definitely, if possible, if these might be the cause of gas formation and, if so, to devise a means of overcoming the difficulty.

The experimental work consisted in trying to reproduce plant conditions in the laboratory in as simple a manner as possible and in such a way as to control at will the various factors involved. All the experiments were carried out in a bomb which consisted of a piece of 4-inch steel pipe closed at each end by a blind flange with a female recess, and clamped together by means of four bolts (Fig. 1). The bomb was provided with an outlet for aqua ammonia, consisting of a steel tube 8 inches long passing through

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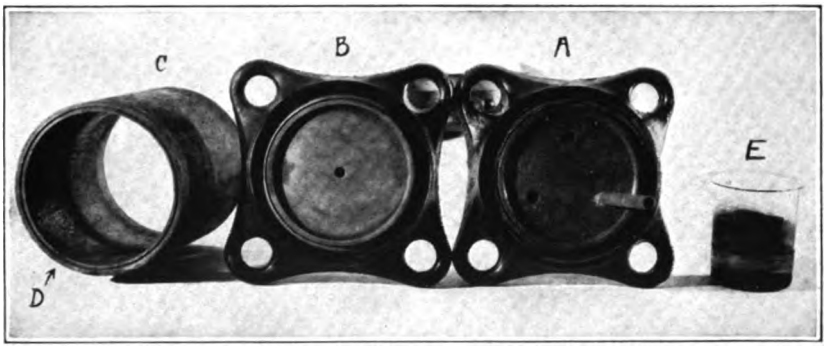

FIG. 1.—*Parts of experimental bomb used in corrosion tests*

A, top flange showing dark coating of oxide formed where gas was in contact with iron; *B*, bottom flange showing bright metal where corrosion took place under the liquid; *C*, pipe clamped between flanges showing corrosion at D where metal was under liquid; *E*, sludge formed in bomb when air was introduced into bomb containing aqua ammonia and the charge was heated.

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the top flange and reaching to the bottom of the bomb. Another tube, which passed through the top flange, was used for taking gas samples. A third outlet was connected to a pressure gage. The bomb, after charging, was heated by means of an electrically heated oil bath, the temperature of which could be kept constant within a few degrees. It would have been more satisfactory in reproducing plant operating conditions, especially where air was intentionally added to the bomb, if the liquid could have been stirred or the bomb rotated; but there were so many experimental difficulties connected with doing this that it was not attempted.

The criterion of the behavior of aqua ammonia and iron toward each other was the formation of noncondensing gas, which collected mostly in the gas phase. Therefore there were in this phase ammonia, water vapor, and any noncondensing gases, such as hydrogen, oxygen, and nitrogen, that might be formed. The noncondensing gases were collected by passing the gas through a capillary tube into a gas burette inverted over sulphuric acid. It was found generally that in the gaseous phase there is practically no action between the various components and the iron. The water, or the oxygen, if air be present, in the gaseous phase reacts with the iron and forms a protective coating of magnetic oxide of iron that seems to remain unaltered until some of the liquid phase touches it; then, if this liquid is of suitable composition, considerable corrosion may take place.

2. EXPERIMENTS WITH DISTILLED WATER

Distilled water, previously heated to remove all dissolved gases, was first charged into the bomb, which was then evacuated and heated to 130° C for a period of 5 days. At the end of the experiment only a very small amount of gas had been formed, as was indicated by an increase of only 4 mm in pressure above the original pressure when the bomb was connected to a mercury gage. When the liquid was withdrawn from the bomb, it was found to be only faintly colored, and there were no signs of corrosion on the interior of the bomb. On heating with pure water to 170° C for a period of 8 days, 80 cc of gas, containing 93 per cent of hydrogen, was collected. The desirability of operating at low temperatures is indicated by the fact that as the temperature is raised the amount of hydrogen produced increases greatly.

The bomb was then recharged with boiled distilled water, and compressed air was forced into it until the pressure was about 50

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pounds. In the course of a few days' heating the oxygen content of the gas was reduced to 10 per cent, the remainder of the gas being nitrogen. The bomb was opened and was found to be badly corroded, and the solution was colored a deep brown and contained considerable precipitate. The conclusion may be drawn that if air gets into a closed system of this kind the oxygen will gradually be removed and the final result will be an atmosphere of nitrogen.

An experiment was performed with water containing 0.1 per cent acetonitrile, the conditions of heating being the same as those when water was heated at 130° C. More gas was produced than with pure water under the same conditions, the pressure in this experiment rising 13 cm above the original pressure. The bomb was attacked, and the solution, although a pale yellow on withdrawal, became darker on standing and gradually changed to a colloidal solution, which finally deposited a yellow precipitate. In all cases where corrosion occurs in the absence of air there is found to be a small amount of ferrous iron in solution. The solution is nearly colorless when first removed, but on standing it becomes a deeper and deeper yellow, owing to the oxidation of the iron from the ferrous to the ferric state. Finally the iron precipitates out as a basic ferric compound.

3. EXPERIMENTS WITH AQUA AMMONIA

The experiments with water were then duplicated with solutions of pure ammonia and with the ordinary concentrated aqua ammonia of commerce, containing about 28 per cent of ammonia. In heating aqua ammonia alone, if it is pure, hydrogen is produced only by heating at high temperatures. In the experiments with either pure water or pure aqua ammonia, the rate of production of the hydrogen diminishes as the period of heating progresses. This is probably due to the fact that before beginning each experiment the interior of the bomb had been polished with emery, while at the end the interior of the bomb was covered with a firm, dense coating of oxide. However, in those cases where impurities such as carbonates or acetates are present, the rate of production of hydrogen shows no such decrease, and upon opening the bomb, the deposit on that part of the bomb in contact with the solution can be easily removed by wiping.

The experiment with air and water was also duplicated with aqua ammonia and air. In this case also the oxygen content

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decreased, but not so rapidly, diminishing under the same conditions from 20 per cent to about 15 per cent. The interior of the bomb was also less attacked. A high-grade sample of aqua ammonia was used, and as the gas was in contact with a saturated solution of ammonia, the solubility of the oxygen in the solution was very greatly decreased. However, the decrease in oxygen content of the gas even in the absence of stirring was very marked.

From the above results it is seen that if air is permitted to enter the system, as may readily happen under certain faulty conditions of operation, there will result finally an atmosphere of nitrogen containing more or less oxygen. It was also found that such impurities as carbonates, acetates, and acetonitrile increase the corrosive power of aqua ammonia at ordinary operating temperatures to such an extent as to readily account for the hydrogen found in many plants.

The two impurities present in aqua ammonia that cause corrosion are, therefore, acetates and carbonates. If acetonitrile is present, it is at once saponified and converted into an acetate by the heated aqua. It is contended that because of its method of manufacture ammonia made from ammonium sulphate is the only ammonia that is free from acetonitrile. No carbon dioxide is present in anhydrous ammonia. However, examination of a number of samples of aqua ammonia showed that carbon dioxide was always present, and the amount may be increased by exposure to the atmosphere. In testing samples of aqua for corrosive properties it was found that from a determination of the carbon dioxide content the action of the aqua could be predicted. Where the content of carbonate is not over 0.003 per cent, the corrosive action is not very marked. And it was also noted that where corrosion once starts it will continue during the life of the charge.

4. DETERMINATION OF CARBON DIOXIDE

From the above evidence it is seen that one of the most important tests of aqua ammonia to be used in an absorption refrigeration machine is the determination of its carbon dioxide content. Since a content of carbon dioxide as low as 0.003 per cent will cause gas formation, a method that will permit the use of large samples, usually 100 g, was adopted.

The sample is weighed in a weight burette and is then slowly added to a glass-stoppered wash bottle containing an excess of sulphuric acid. A current of air free from carbon dioxide is

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passed continually through the wash bottle and into a special wash bottle containing barium hydroxide solution. This latter wash bottle is described by E. R. Weaver and J. D. Edwards (J. Ind. Eng. Chem., 7, p. 534; 1915). When all the ammonia solution has been added, the solution is boiled for a few minutes and the air is allowed to pass through until all the carbon dioxide has been swept over into the wash bottle. The barium carbonate thus formed is washed free from barium hydroxide, dissolved in hydrochloric acid, and finally precipitated as barium sulphate. From the weight of barium sulphate thus obtained the percentage of carbon dioxide is easily calculated. Some of the results obtained are as follows:

TABLE 1.—Carbon Dioxide Determination

Sample	CO_2	
	Per cent	Per cent
C. P. reagent	0.0015	0.0016
Technical sample	.0026	.0026
Technical sample	.0075	.0071
Tap water	.0052	.0052
B. S. plant No. 1[a]	.066	.067
B. S. plant No. 2[a]	.087	.087
Calcium carbonate C. P.[b]	44.0	44.3

[a] 50 g sample used. [b] 0.1 g sample used.

5. METHODS OF PREVENTING GAS FORMATION

Among the substances which have been proposed for preventing the corrosion of iron in general is potassium dichromate solution. The authors therefore conducted experiments to determine whether a dichromate would be useful in ammonia systems. As these experiments were performed during the period of the war, sodium dichromate was used and was found to be very satisfactory. Solutions of aqua ammonia were made up to contain the various corrosive constitutents and in addition approximately 0.2 per cent by weight of commercial sodium dichromate. In all cases the amount of gas produced was very small, and less than that given by the best sample of ammonia solution that had been tested without the addition of dichromate. The solution after withdrawl from the bomb had a green color which did not change on standing. There was always a precipitate present which varied from a grayish green when pure samples of aqua ammonia were used to dark brown when the dichromate was added to the contents of a bomb in which corrosion had already

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taken place. The addition of dichromate always stopped the gas formation, and upon opening the bomb the characteristic hard dull coat was observed. From these experiments in the laboratory it was decided to try the addition of sodium dichromate to the ammonia in a plant which was giving trouble on account of foul gas formation.

Much trouble had always been experienced in the operation of the 30-ton Carbondale absorption plant at the Bureau of Standards. Gas was being formed continually, and this caused large losses of ammonia in purging the system of foul gas, as well as losses due to leakage caused by the high-working pressure necessary because of the presence of the noncondensing gas. The ammonia charge at this plant became depleted during the winter 1916–17, and it was found necessary to partially recharge it. Analysis was made of the ammonia solution that still remained in the machine, and it was found to contain 19 per cent of ammonia and about 0.066 per cent of carbon dioxide. In recharging this system tap water, containing about 0.0052 per cent total carbon dioxide, and anhydrous ammonia were used. After recharging the system and operating for a few months, the carbon dioxide content had increased to 0.087 per cent and there was considerable corrosion and gas formation. The analyzed samples of liquor when tested in the experimental bomb were found to cause a great deal of corrosion and gas formation, but when sodium dichromate was added gas formation was stopped.

In June, 1917, an amount of sodium dichromate, equal to 0.2 per cent of the total weight of the aqua ammonia charge, was added in the form of an aqueous solution to the plant. No trouble has since been experienced with the plant from gas formation. At one time when the engineer reported that considerable gas was being formed in the plant, the gas was found upon analysis to consist of over 95 per cent nitrogen with a few per cent of hydrogen and a small amount of oxygen. It was soon found that this gas was caused by leaks around the piston rod of the ammonia pump. On replacing the worn-out rod gas troubles ceased and have not been experienced since. This plant is now giving better service than for many years past.

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6. SUMMARY

1. The noncondensible gases found in ammonia absorption refrigeration machines are due to either or both of two causes, namely, (*a*) leaks of air into the system and (*b*) the corrosive action of the ammonia liquor on the metal of the plant.

2. When the foul gas is mainly nitrogen, the gas is derived from air that has leaked into the system, and leaks should therefore be sought. The oxygen in the air is very quickly used up and so will be present in only a very small percentage of its original amount. If the foul gas is hydrogen, the cause is corrosion by the ammoniacal liquor. A gas containing both nitrogen and hydrogen shows both causes to be present.

3. If a solution of sodium or potassium dichromate is added to the generator charge so that the charge in the generator will contain the salt to the extent of 0.2 per cent by weight, all foul gas formation from the corrosive action of the ammonia charge will be stopped. It is recommended that the dichromate be added in all cases, as it has been found that its presence decreases the very small amount of gas caused by even the highest grade ammonias.

4. A method is given for the quantitative estimation of carbon dioxide in ammonia.

WASHINGTON, July 19, 1920.

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TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 181

COLORED WALL PLASTER

BY

WARREN E. EMLEY, Chemist
CHARLOTTE F. FAXON, Laboratory Assistant
Bureau of Standards

DECEMBER 15, 1920

PRICE, 5 CENTS

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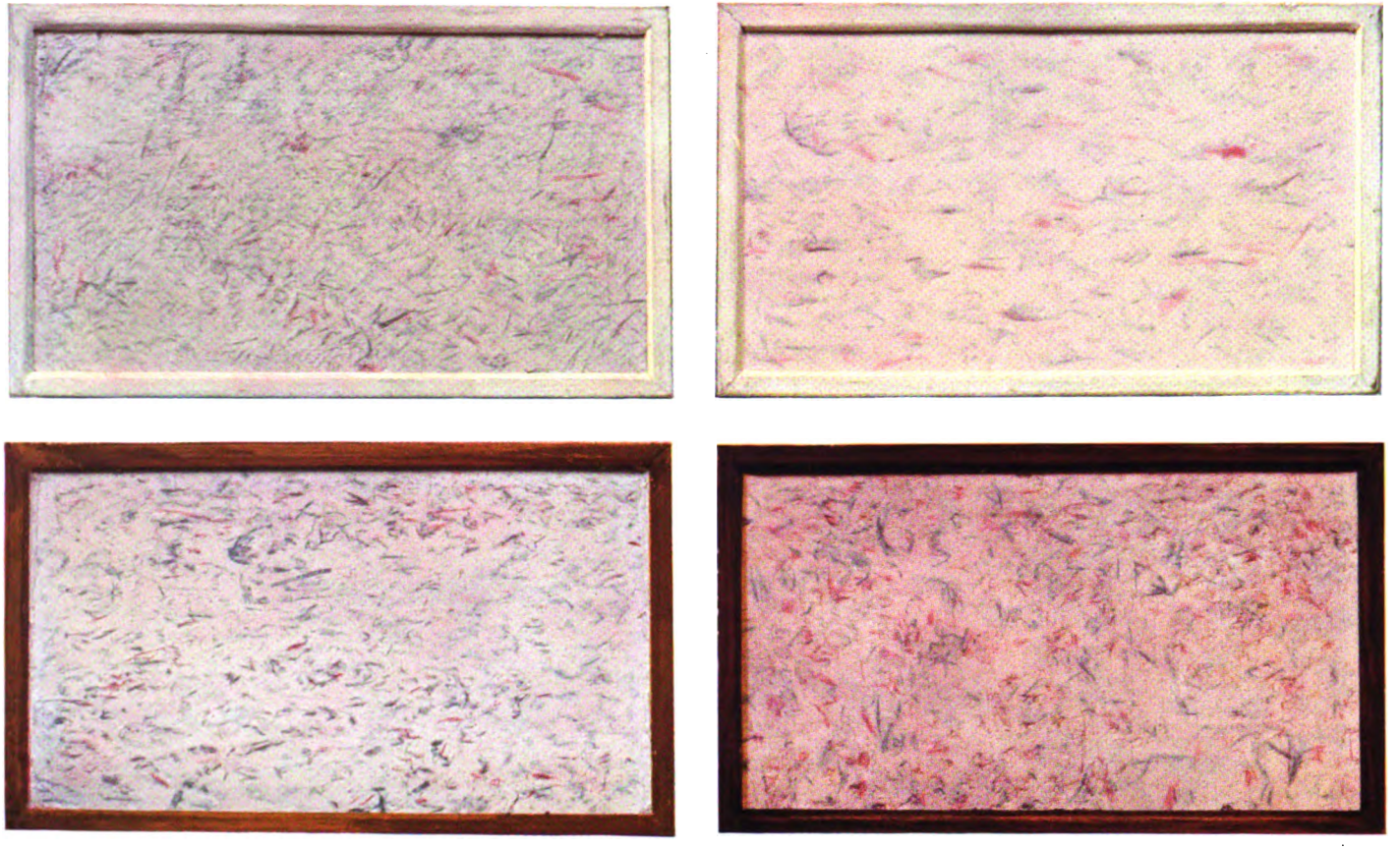

FIG. 1.—*Examples of the new type of colored wall plaster.*

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COLORED WALL PLASTER

By Warren E. Emley and Charlotte F. Faxon

The natural color of a wall plaster is white, or nearly so. For many years it has been customary to accept this white color as inevitable and to dismiss the subject without further consideration.

The quality of a plaster is indicated by its natural color in a way which is strikingly analagous to the method used for classifying diamonds. The best plasters are slightly tinted with blue; pure white plasters are of excellent quality, and the poorest grades have a buff or reddish tinge. This fact has led the plasterer and the building contractor to look with favor upon a white or bluish-white plaster because these colors are usually accompanied by excellence in quality.

On utilitarian grounds, plaster has established its position in the building industry. Its fire resistance, its heat insulating, and its acoustical properties are of primary importance. The appearance of the plaster, while admittedly and rightfully of secondary importance, is receiving more and more attention from progressive architects and builders.

The white color of plaster is sometimes objectionable in that it does not fit in with the artistic scheme which the architect is trying to evolve. It is true that the plaster may be covered with kalsomine or wall paper, but it seems to be worth while to attempt to make the plaster of the desired color in the first place.

Several attempts have been made to produce colored plaster, but the method has not proven sufficiently satisfactory to warrant its general adoption. It involves the use of a mineral pigment, which is mixed with the plastering materials. A buff or red can be obtained with oxides of iron; a gray with lampblack. It has been found extremely difficult to mix the pigment with the plaster in such a way that two batches have exactly the same tint. Deep shades of color require the addition of so much pigment that the strength of the plaster is impaired. A plaster made by this method has a dead color, which is desirable when the plaster is considered as a background for other decorations, but which has little artistic value of itself.

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Some years ago J. J. Earley developed on a commercial scale the manufacture of colored concretes by the exposed aggregate method. Many examples of the beauty of such concretes and of the flexibility of the process can now be seen in buildings, walls, and pavements of Washington. Instead of the usual crushed stone or gravel which is used as an aggregate in concrete, he uses crushed stone which is prepared from naturally colored rocks, carefully selected and mixed to produce the desired artistic effect. After the concrete has set, the surface is removed and the colored aggregate thereby exposed. At Mr. Earley's suggestion this same principle has been applied to the manufacture of a colored wall plaster.

The first step was the selection of the proper aggregate. Fortunately, this offered no difficulty. There has been on the market for many years a material known as "gypsum wood-fibered plaster." This consists of approximately 85 per cent of calcined gypsum (plaster of Paris), 14 per cent of some material such as lime or clay added to give it the necessary plasticity, and 1 per cent by weight of wood fiber. It is one of the many commercial forms of wall plaster, and its position in the trade is so well established that tentative specifications for it have been adopted by the American Society for Testing Materials.

This plaster is usually designed to be used without sand, the wood fiber being considered as an aggregate. The fiber is quite similar to the ground wood used in making newspaper, except that the staple is a little longer.

The proposed process consists briefly of dyeing the fiber the desired color with aniline dyes, mixing it with the other ingredients, applying the plaster to the wall in the usual way, and then, after the plaster has set, removing its surface in such a way as to expose the colored fiber.

The first difficulty encountered arose from the lack of literature describing the behavior of aniline dyes on wood. Most of our experimental work was devoted to this phase of the subject—the testing of a large number of dyes. These tests were conducted by exposing half of a sample of dyed wood fiber mixed with the other ingredients of a colored plaster to direct sunlight for six weeks. The other half of the same specimen of plaster was kept covered. If, at the end of six weeks, one could not distinguish between the two halves of the sample, the dye was pronounced satisfactorily fast to light. Fastness to water is not essential; in fact, it is usually desirable that the dye shall "bleed" a little.

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The color runs into the plaster surrounding the fiber, thereby making the fiber appear to be larger than it is and making it possible to obtain the desired effect with the use of less fiber than would be possible if the dye were fast to water.

It was found that wood fiber acts quite similarly to jute in taking up dyes. Any one of the classes of dyes can be used—silk, wool, cotton, or direct. A dye which is fast to light on the fiber on which it is intended to be used will usually be fast to light on wood fiber. If the dye is fast to water, it can sometimes be made to bleed slightly by drying the dyed fiber, without washing it. As a general rule, a small amount of alum added to the dye bath answers the purpose of a mordant. The heterogeneous character of the wood prevents the dye from being absorbed evenly. As will be shown in the following discussion, this is an advantage rather than a detriment, so that no leveling agent is necessary.

A general method of procedure may be illustrated as follows: In an indefinite quantity of water, dissolve an amount of dye equal to 3 per cent of the weight of the fiber. Heat the solution nearly to boiling and add the fiber. Add about 2 per cent of alum (aluminum sulphate) based on the weight of the fiber. Digest hot for about one hour and let it cool. Remove the fiber and dry it (usually without washing) in a drying oven. These directions are to be taken in a very general sense. Changes in them, to suit the different dyes, will be suggested by the directions given by the dye manufacturers, by standard textbooks, and by experience.

The kinds of dyes which may be used are illustrated by the following list of a few of the many dyes we have found satisfactory: Alizarine cyanine green G extra, alizarine cyanine yellow, alizarine cyanine irisol, crocein scarlet 3 B, rhodamine B extra, sulphur blue, diamine brown, diamine rose, naphthol orange, beta naphthol yellow, flavopurpurine, alizarine tartrazine, chloramine brown. Obviously any desired color can be obtained.

It is to be understood that these colors have proven fast to light when tested by the laboratory method described above. Before attempting any commercial application, it would, of course, be wise to determine the permanency of the dyes which it is proposed to use. This information may be obtainable from the dye manufacturers, or it may be necessary to learn from experience. Attention is called to the fact that the color need not be perfectly fast to light to give satisfactory service. If a wall is evenly exposed, so that all parts of it fade at the same rate, the fading is not a serious defect. There is also the probability that the walls will be redecorated every few years to suit the tastes of new occupants of the building.

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There are a great many pure chemical compounds which have definite colors in and of themselves. A massive piece of such a compound will appear in its true color, irrespective of the viewpoint of the observer. However, most of the materials with which we are familiar are not simple substances, but are mixtures of several ingredients. This is particularly true of wall plaster. If the different ingredients have different colors, as is usually the case, then the color of the mixture will depend upon the relative amounts of the ingredients, the relative sizes and positions of the particles, and the viewpoint of the observer. A plaster which appears gray on casual observation may, on close inspection, be found to consist of a number of black specks embedded in a white matrix. If the black specks are large enough to be easily visible, then the plaster is no longer gray, but is a mottled black and white. It will be noted that the term "easily visible" defines the position of the observer. The mottled effect, which is pleasing to the eye, is obtained only when the black particles are uniformly distributed; if they are collected in large aggregates, the effect is blotchy and displeasing.

While fibers of one color only can be used, better effects can be obtained by the use of two or more different colors. In this way, the predominating color can be made more agreeable and satisfying by the use of supporting colors. Spots of the principal color should be distributed uniformly (but not in regular pattern) throughout the matrix. They should be large enough so that their identity can be distinguished without effort. Each spot should be set off by surrounding it with a number of small spots of one or more secondary colors. These should be so small that they can not be distinguished except by close inspection. The effect of the whole can be improved by giving the surface a rough texture.

Of course, due care must be taken that only those colors are used together which harmonize well with each other.

Plasters colored by this method will have "life." By this it is meant that the color of the plaster is more or less dependent upon the position of the observer; the color changes as he moves. Suppose, as the simplest case, that a plaster contains fibers of two principal, but no secondary, colors. When viewed from one direction, it is probable that one color would be reflected much more than the other and would become dominant. Viewed from another angle, the reverse might be the case. Close inspection would reveal the two colors equally. At a sufficient distance, the reflected colored lights would be superimposed, giving rise to an

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entirely different color, or even to gray if the original colors are complementary.

A similar live effect can be produced by a rough surface. The protuberances cast shadows on the depressions, and thus produce light and dark tints. The intensity and the apparent magnitude of these shadows depend upon the position of the observer, and seem to vary as he moves. The life of such a surface, and its pleasing effect, is dependent upon its texture.

The above statements form a brief outline of the principles underlying the manufacture of this type of colored plaster. It will be noted that the effect desired may be described as a "uniform lack of uniformity." It is desirable that the wood fiber shall not take up the dye evenly. The principal color can be made predominant either by using a large quantity of fiber or else by using a dye which will "bleed." For the secondary colors, the fiber should be quite small and the dye fast to water, or, if the dye will bleed, the fiber may be replaced by sawdust.

The desired proportions of the different colored fibers are mixed with the calcined gypsum and other ingredients to form a gypsum wood-fibered plaster of the usual composition. This is shipped dry, mixed with lime putty and water, and applied in the usual way. Up to this point the only novelty in the manufacture of a colored plaster lies in the use of a dyed wood fiber instead of the naturally colored material. Of course care must be exercised in the selection of colors which will blend harmoniously, and in the admixture of the proper proportions of primary and secondary colors.

The desired texture of the finished surface may call for a variation in the quantity of fiber used. It will be recalled that commercial wood-fibered plaster contains about 1 per cent of fiber by weight. This is equivalent to about 15 per cent by volume. About 1 part of fiber to 1 of plaster, by volume, is the maximum amount which can be used if the surface is to be troweled smooth. If, however, a rough surface is desired, the quantity of fiber may be increased; 2 parts of fiber to 1 of plaster can be used by patting it into place.

A rough texture is produced by brushing the plaster just as it sets. This removes the gypsum from the surface and exposes the colored fibers. Differences in the quantity of fiber and in the method of brushing permit the production of a great variety of textures. We have developed three types of texture which are illustrative of variations obtainable. The "smooth finish"

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is obtained by the use of a plaster containing 0.3 to 1 part by volume of fiber to 1 part of plaster. The plaster is troweled to a smooth surface in the usual way. Since this surface is not brushed, and the fiber is not exposed, it is necessary to use dyes which will bleed rather profusely. Otherwise the color will not be sufficiently pronounced. Just before the plaster has set, it may be brushed with a fiber scrubbing brush, taking care that the marks made by the brush are all either horizontal or vertical. This produces an "organdie" finish. For the "tapestry" finish, we use 2 parts of fiber to 1 of plaster. This is patted into place but can not be troweled. Just before the plaster sets it is stippled with a wire brush. The resultant surface is very rough and bears some resemblance to tapestry. These types are illustrated in the colored photograph which accompanies this paper.

It was found that a slight efflorescence appeared during the drying of the plaster, which marred the brilliance of the colors. This was overcome by washing the plaster with soap and water. The soap can be applied just before the plaster is finally troweled down, or the plaster may be allowed to harden and dry before scrubbing it. A thick solution of strong laundry soap in hot water is recommended. Apparently the soap enters into chemical reaction with the lime in the plaster forming a calcium resinate or similar gelatinous compound, which effectually closes the pores of the plaster and prevents further efflorescence.

This method of scrubbing the plaster with soap and water affords a housewife a means of brightening the walls whenever necessary.

No matter how pleasing the appearance of a wall may be, it is probable that a change in the decorative scheme will sometime be desired. When this occurs, it is only necessary to remember that a colored wall plaster is essentially the same as any other wall plaster. It may be covered with another coat of plaster, either colored or white, or it may be painted or papered in the usual way.

In conclusion, it may be stated that a method has been developed for producing a colored wall plaster of any desired color or texture. Effects can be produced with this plaster which are not attainable with either paint or wall paper. A wall finished in this plaster can be washed when the colors become dull or soiled, or it can be redecorated in the same way as any other plastered wall.

WASHINGTON, August 16, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 182

EFFECT OF REPEATED REVERSALS OF STRESS ON DOUBLE-REINFORCED CONCRETE BEAMS

BY

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DECEMBER 20, 1920

PRICE, 15 CENTS
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By W. A. Slater, G. A. Smith, and H. P. Mueller

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I. INTRODUCTION

1. GENERAL STATEMENT

In a ship at sea the change from "hogging" to "sagging" may occur 10 to 15 times per minute. This means a large number of reversals of stress during the life of a ship, and it means that both the deck and the bottom of the ship are required to withstand tensile and compressive stresses alternately. The effect of these reversals of stress has a bearing upon the use of reinforced concrete in lieu of wood and steel in the construction of ships, and the investigation of this effect is a matter of importance.

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Although earlier tests[1] have been reported showing the effect of repeated loads in one direction on a concrete beam, none are known to have been made in which a concrete member has been subjected alternately to compressive and tensile stresses large enough to crack the concrete. Assuming that the action above described would sometime produce cracks in the structure of a concrete ship, the possibility was foreseen that the repeated opening and closing of the cracks might produce a grinding action on the surface of the fracture which would result ultimately in disintegration. This was one of the important features considered in the tests here reported. This investigation was undertaken by the Bureau of Standards at the request of the concrete ship section of the Emergency Fleet Corporation.

Acknowledgment is made to A. S. Merrill, engineer of the Bureau of Standards, for his editorial work upon this paper and for many valued suggestions and criticisms for its improvement.

2. SCOPE OF TESTS

The scope of this investigation was intended to bring out only the outstanding features of the behavior of reinforced concrete beams subjected to repeated reversal of stress. The original program was modified somewhat after the results of the first tests were obtained.

In the investigation only five beams were tested. One of these five beams had not failed at the time that it became necessary to discontinue the testing operations. However, since this beam had undergone a large number of repetitions of a load which produced approximately the design stresses, it furnishes information of some value.

Although the tests which have been carried out are considerably smaller in number than was planned, the impossibility of completing the program in the near future makes it desirable that the results which have been obtained be presented at the present time.

[1] J. L. Van Ornum, The Fatigue of Concrete, Trans. Am. Soc. C. E., 58, p. 294, 1907; W. K. Hatt, Notes on the Effect of Time Element in Loading Reinforced Concrete Beams, Proc. Am. Soc. Test. Mat., 7, p. 421, 1907; H. C. Berry, Some Tests of Concrete Beams under oft repeated Loading, Proc. Am. Soc. Test. Mat. 8, p. 454, 1908.

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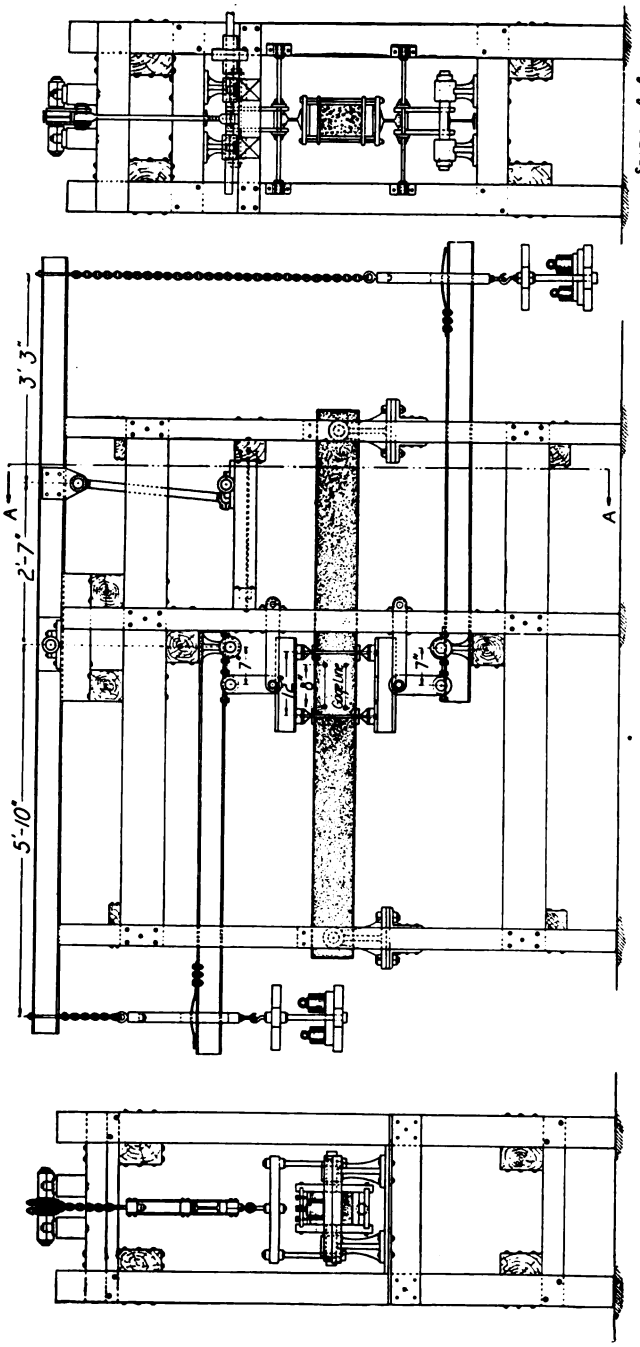

FIG. 1.—*Reversal-of-stress testing machine*

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II. TESTING APPARATUS

1. TESTING MACHINE

The machine designed for these tests applied a load alternately downward and upward at a rate of approximately 17 cycles[2] per minute. Three units of the machine were built so that three beams might be under test at the same time.

The testing machine is shown in Fig. 1, and consisted essentially of a wooden frame with a system of levers so arranged and operated that a load applied at the end of one of the main levers produced a load ten times as great on the concrete test beam. One main lever applied the load in the upward direction and the other main lever applied it in the downward direction. When one load was being applied, the load in the opposite direction was being removed. Slightly before one load had been entirely removed the full load in the opposite direction had been applied.

During the test the beam was supported at sections 4 inches from each end by means of steel yokes, shown in Fig. 1. Each yoke was supported at mid height by a shaft which was free to turn as the beam deflected. Since the supporting shaft was approximately at the neutral axis, the change in the distance between the point of support, due to changing the curvature of the beam, was negligible.

The load was transmitted from each main lever through a link to a rail section, as shown in Figs. 1 and 2. By means of the rail section the load was applied to the beam at points usually 6 inches on either side of the center of the span both on the top and on the bottom of the beam, but in testing beam 5L1 the knife-edges were placed 12 inches either side of the center. The knife-edges through which the load came upon the beam were held in place by means of a yoke around the beam at each point of application of load

When tests were being conducted, the units were driven by means of a shunt-wound electric motor developing 3-horsepower at 1700 revolutions per minute. The shaft of the motor is directly connected to a worm-gear reducing unit. An individual shaft, clutch, and pulley arrangement enables each unit to be operated independently. The walking beam, which applies the load to the main levers, is driven by means of a connecting rod from a crank attached to the individual driving shaft of each unit. A

[2] The term "cycle," as used in this report, refers to one complete revolution of the crank shaft of the testing machine. Each cycle involved one application of the load downward and one application of the load upward.

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control rheostat regulates the speed of the motor at different loads and is the means of obtaining the same speed whether running with one unit or all units in operation. For a part of the time that the tests were in progress the machines were run continuously day and night.

Fig. 3 shows the position of the three units of the machine relative to each other and to the driving motor. Fig. 4 is a side view of a beam in one of the units of the testing machine.

2. STANDARDIZATION OF TESTING MACHINE.

A standard 6-inch I-beam was used to rate the reversal-of-stress testing machine. The I-beam was first standardized in a 200 000-pound Olsen testing machine, using an 8-foot span and 2-point loading, 6 inches either side of mid span. Loads were applied in increments of 500 pounds, as measured on the scale beam of the Olsen machine, and the corresponding deflections were measured at mid span. Readings of deflection were also noted as the load was removed. The beam was then inverted and loaded again, and it was found that the load-deflection curve agreed closely with that for the beam loaded in the previous position. The extreme fiber stress did not exceed the elastic limit of the steel. The I-beam was then transferred to the reversal-of-stress machine and loaded with the same spacing of load points and supports as that employed in the Olsen testing machine and in the tests of the reinforced concrete beams.

From the deflection curve obtained from the test in the Olsen machine and the deflection curve from the loading in the reversal-of-stress machine, curves were drawn which gave the relation between the applied load and the load on the weight carrier. It was found that the equations of these curves varied slightly for the different units. The equations found were corrected for the weight of the beam and for certain parts of the loading apparatus and were used in determining the loads on the test specimens. The equations are as follows:

Unit 1 upward $W = 2088 + 10.03\ W_1 - \frac{4}{7} W_2 - W_3$.
Unit 1 downward $W = 2070 + 9.79\ W_1 + \frac{4}{7} W_2 + W_3$.
Unit 2 upward $W = 2078 + 9.88\ W_1 - \frac{4}{7} W_2 - W_3$.
Unit 2 downward $W = 1990 + 10.05\ W_1 + \frac{4}{7} W_2 + W_3$.
Unit 3 upward $W = 2070 + 9.83\ W_1 - \frac{4}{7} W_2 - W_3$.
Unit 3 downward $W = 2000 + 9.41\ W_1 + \frac{4}{7} W_2 + W_3$.

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FIG. 2.—*View of unit No. 1 of reversal-of-stress testing machine*

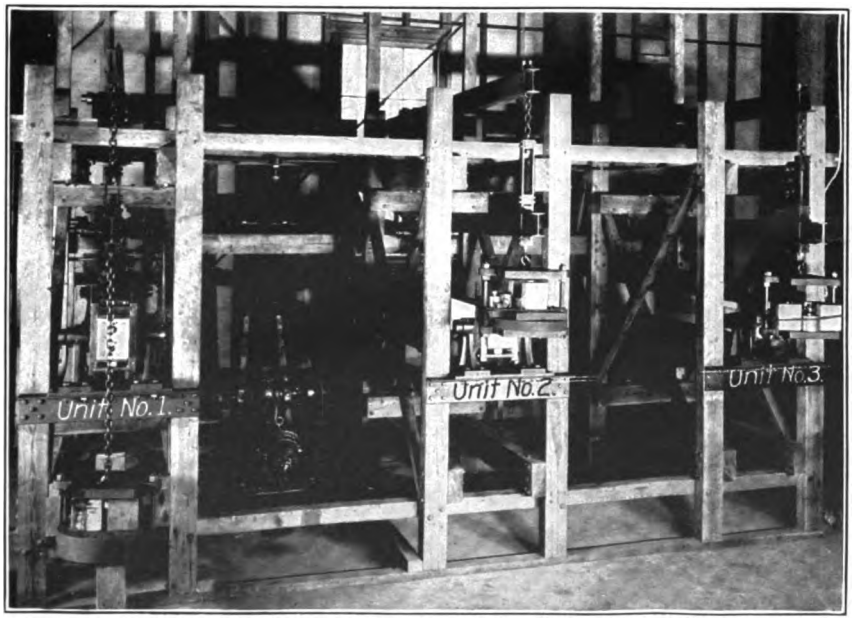

FIG. 3.—*View showing the three units of reversal-of-stress testing machine*

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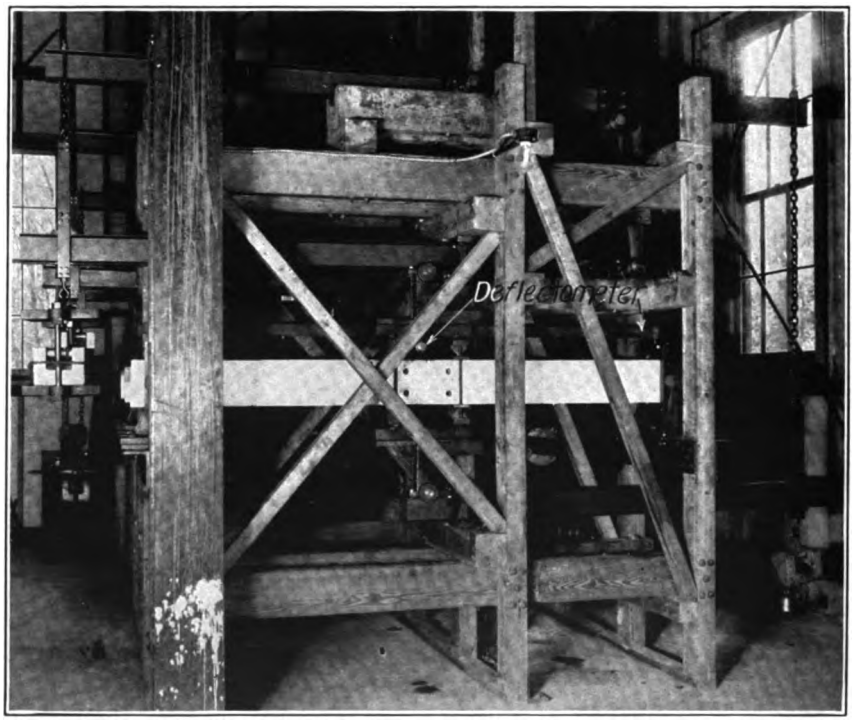

FIG. 4.—*View showing beam 5C1 in unit No. 3 of testing machine with deflectometer in position*

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In these equations W is the load upon the test beam, W_1 is the load applied to the weight carrier, W_2 is the weight of the test specimen, and W_3 is the weight of the yokes and T-rails used for applying the load.

It will be seen that for all the units the load without weights being added to the weight carriers was about 2000 pounds. In testing beam 5C1 it was desired to apply only 670 pounds upward. This necessitated removing the weight carrier at one end of unit No. 3 and the attaching of small weights to the main lever. This may be seen at the right of the view in Fig. 4.

3. MEASURING INSTRUMENTS

A Berry strain gage, having an 8-inch gage length, was used to measure the deformations in the steel reinforcement. The multiplication ratio of this strain gage was 4.94. Readings on a gage line on an unstressed bar of structural steel were used as a basis for correcting all strain-gage readings on the test specimens.

For measuring deflections a removable deflectometer was used. It consisted of a wooden bar having fixed trammel points 8 feet apart and having an Ames-gage micrometer mounted at its center. The wooden bar was supported on the steel yokes at the ends of the test beam in the same position for consecutive observations. The difference in the deflections at the center of the span was shown by the Ames gage. Readings were made directly to the nearest 0.001 inch. To provide for the exact placing of the deflectometer, one end was held in a gage hole on one supporting collar, while the opposite end was held on the other collar in a groove cut parallel to the axis of the beam. This allowed compensation for the upward and downward movement of the points of support of the deflectometer which took place as the beam deflected. The deflectometer may be seen on the test specimen in Fig. 4.

To obtain the corrections necessary to be applied to the deflectometer readings to compensate for the unavoidable errors caused by vibrations, change in temperature, or other uncontrollable or accidental movement of the gage, standard readings of the deflectometer were taken on the top flange of a 24-inch steel I-beam which was not under load.

Ames-gage micrometers were used to measure the slip of the ends of the bars in beam 5L1. The reinforcement in this beam consisted of two 1¼-inch plain round bars placed one in the top of the beam and the other one in the bottom, as shown in Fig. 5. Holes were drilled and tapped in the ends of these bars and short pieces of steel were screwed in place. The gages were clamped

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to these extension rods with the plungers bearings against the ends of the beam. Gage No. 1 was mounted at the east end of the top bar, gage No. 2 at the east end of the bottom bar, gage No. 3 at the west end of the top bar, and gage No. 4 at the west end of the bottom bar.

For each unit the number of cycles of load was shown by a revolution counter connected to the main shaft of the unit.

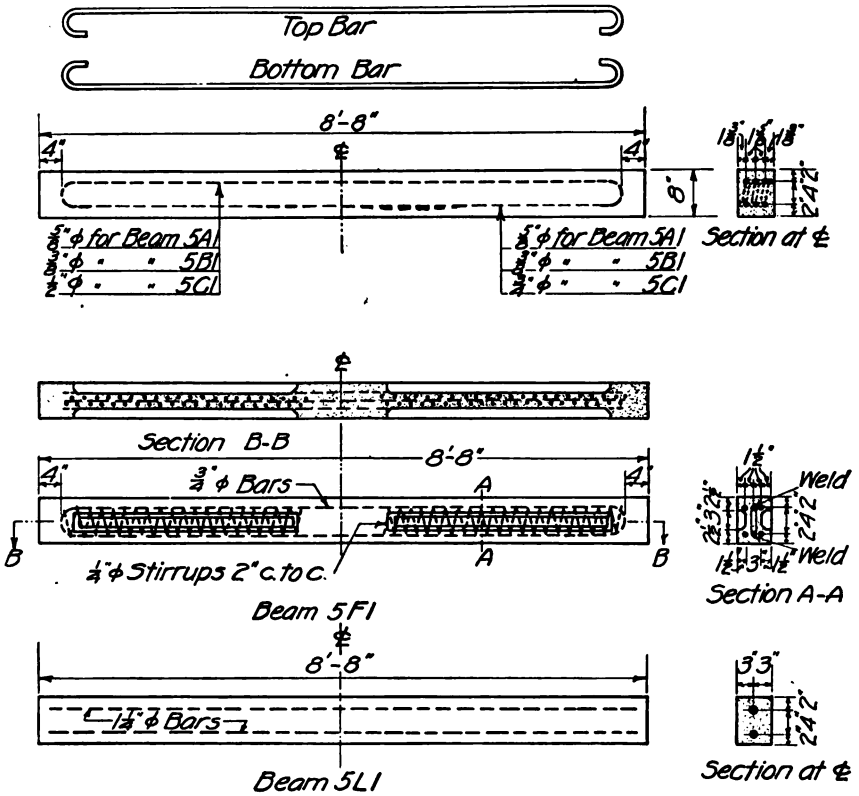

FIG. 5.—*Dimensions and reinforcement of test specimens*

III. TEST SPECIMENS AND TESTING

1. DESCRIPTION OF TEST BEAMS

The beams were 8 feet 8 inches long, 6 inches wide and 8 inches deep, and the test span was 8 feet. The top bars were placed generally 2 inches below the top surface, and the bottom bars 2 inches above the bottom surface. The cross section was rectangular for all of the beams except 5F1. That beam was I-shaped in cross section, both flanges being 6 inches wide and 2½ inches deep; the web was 3 inches thick, and the total depth of the beam was 8 inches. At the ends, where beam 5F1 was supported, and

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between the load points the web was filled out to the full 6-inch width of the flange in order to avoid serious secondary stresses due to the concentration of load at these points. Beam 5F1 was made with an I-shaped cross section in order to develop a high shearing stress in the web without affecting appreciably the resistance to tensile and compressive stresses.

The reinforcing bars in all of the beams except 5L1 had semicircular hooks on the ends to furnish anchorage. Beam 5L1 was reinforced with straight bars in order to study the effect of repetition of high bond stresses on the end slip of unanchored bars.

The depth of embedment of the reinforcing bars was designed to be 2 inches in all cases. In general, it was desired to develop high compressive stresses in the concrete, and this could be accomplished in these tests only by placing the bars farther from the compression surface of the beam than otherwise would have been done. Fig. 5 shows the arrangement of the reinforcement in the beams.

Table 1 shows the amount of reinforcement used, the loads applied, and the computed stresses for all of the test specimens.

TABLE 1.—Schedule of Beams for Reversal-of-Stress Tests

Beam No.	Reinforcing bars				Load applied	
	Top		Bottom			
	No.	Diameter	No.	Diameter	Upward	Downward
		Inches		Inches	Pounds	Pounds
5A1	3	⅝	3	⅝	5000	5000
5B1	3	⅜	3	⅜	2380	2380
5C1	3	½	3	¾	670	3630
5F1[a]	3	¾	3	¾	5000	5000
5L1	1	1¼	1	1¼	6000	6000

Beam No.	Computed stresses						Remarks
	Tension in steel		Compression in steel		Compression in concrete		
	Top	Bottom	Top	Bottom	Top	Bottom	
	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²	
5A1	23 590	23 590	4293	4293	2020	2020	
5B1	28 800	28 800	−112	−112	1538	1538	
5C1	4 800	12 050	480	5080	311	1360	
5F1	16 750	16 750	5360	5360	1641	1641	I section; computed shear 175 pounds per square inch.
5L1	19 100	19 100	5900	5900	1835	1835	Computed bond stress 161 pounds per square inch.

[a] Web reinforced with one-fourth-inch wire loops, 2-inch pitch.

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2. MATERIALS

A Portland cement was used which passed the tests prescribed in the United States Government Specification for Portland Cement [3] and which appeared to be of good quality in all respects.

Potomac River washed sand and gravel were used. The sand was graded in size, and was all less than one-fourth inch. The gravel was graded in size from one-fourth to one-half inch. All of the aggregate appeared to be clean, and sections sawed from some of the beams showed a good gradation of the materials, as shown in Figs. 7 and 8.

The reinforcing steel used in the test beams was purchased in the open market. Coupons for testing were taken from the steel before the beams were made. After the reversal of stress tests had been completed additional specimens of steel were taken from the broken beams and were subjected to chemical analysis and tests of their physical properties. The results of the physical tests on the individual test specimens are given in Tables 2 to 5, and the chemical analyses are given in Table 6. Eighteen specimens of the steel taken from the tested beams passed the cold bend test by being bent through an angle of 180° without any sign of fracture.

TABLE 2.—Physical Properties of Reinforcing Steel of Beam 5A1

SAMPLES TAKEN BEFORE MAKING THE BEAM

Sample No.	Location in beam	Original diameter	Yield point	Ultimate strength	Elongation in 8 inches	Reduction of area
		Inch	Lbs./in.²	Lbs./in.²	Per cent	Per cent
1		0.632	37 400	51 500	(a)	46.5
2		.629	38 500	52 700	12.5	42.0
3		.635	36 400	50 500	24.0	46.0
Average		.632	37 400	51 600	18.3	44.8

SAMPLES TAKEN FROM BEAM AFTER TEST

1	Top, north	0.637	35 950	49 600	24.8	50.4
2	Top, middle	.621	39 600	53 850	29.5	55.8
3	Top, south	.632	40 500	51 500	(a)	43.5
4	Bottom, north	.624	43 620	55 650	21.3	52.0
5	Bottom, middle	.631	38 400	52 150	25.8	43.3
6	Bottom, south	.626	39 650	53 950	20.2	57.3
Average		.629	39 620	52 780	24.3	50.4

[a] Broke outside gage length.

[3] Circular of the Bureau of Standards, No. 33; 1917.

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TABLE 3.—Physical Properties of Reinforcing Steel of Beam 5B1

SAMPLES TAKEN BEFORE MAKING THE BEAM

Sample No.	Location in beam	Original diameter	Yield point	Ultimate strength	Elongation in 8 inches	Reduction of area
		Inch	Lbs./in.²	Lbs./in.²	Per cent	Per cent
1		0.381	38 830	51 890	22.0	38.0
2		.379	40 160	53 060	17.9	39.0
3		.378	44 120	50 490	13.3	16.6
Average		.379	41 040	51 810	17.7	31.2

SAMPLES TAKEN FROM BEAM AFTER TEST

1	Top, north	0.378	47 500	60 200	12.5	21.0
2	Top, middle	.377	42 350	53 950	16.4	27.5
3	Top, south	.388	40 400	52 400	18.3	34.9
4	Bottom, north	.386	39 300	55 150	18.0	25.1
5	Bottom, middle	.384	38 500	50 950	22.3	28 8
6	Bottom, south	.386	35 600	52 650	16.7	41.6
Average		.383	40 610	54 220	17.4	29.8

TABLE 4.—Physical Properties of Reinforcing Steel of Beam 5F1

SAMPLES TAKEN BEFORE MAKING THE BEAM

Sample No.	Location in beam	Original diameter	Yield point	Ultimate strength	Elongation in 8 inches	Reduction of area
		Inch	Lbs./in.²	Lbs./in.²	Per cent	Per cent
1		0.757	43 000	58 800	(a)	62.0
2		.746	41 100	60 000	28.0	55.0
3		.755	40 600	58 310	28.8	63.5
Average		.753	41 570	59 040	28.4	60.2

SAMPLES TAKEN FROM BEAM AFTER TEST

1	Top, north	0.755	41 500	60 500	28.5	65.9
2	Top, south	.754	42 500	61 300	35.5	62.9
3	Bottom, north	.750	44 100	62 000	24.0	61.5
4	Bottom, south	.755	38 600	60 700	48.0	59.4
Average		.754	41 680	61 130	34.0	62.4

[a] Broke outside gage length.

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TABLE 5.—Physical Properties of Reinforcing Steel of Beam 5L1

SAMPLES TAKEN BEFORE MAKING THE BEAM

Sample No.	Location in beam	Original diameter	Yield point	Ultimate strength	Elongation in 8 inches	Reduction of area
		Inches	Lbs./in.²	Lbs./in.²	Per cent	Per cent
1		1.270	37 000	66 200	49.5	49.8
2		1.270	38 600	66 500	48.5	49.8
Average		1.270	37 800	66 350	49.0	49.8

SAMPLES TAKEN FROM BEAM AFTER TEST

1	Top	1.272	38 500	66 600	50.5	52.6
2	Bottom	1.268	39 400	66 800	49.0	50.5
Average		1.270	38 950	66 700	49.8	51.6

TABLE 6.—Comparison of Representative Physical and Chemical Tests of Reinforcing Steel Used in Concrete Ships and Reversal-of-Stress Beams

SAMPLES TAKEN FROM REVERSAL-OF-STRESS BEAMS AFTER TEST

Sample taken from—	Physical tests					Chemical contents				Grade
	Diameter	Yield point	Ultimate strength	Elongation in 8 inches	Reduction of area	Carbon	Manganese	Phosphorus	Sulphur	
	Inches	Lbs./in.²	Lbs./in.²	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	
Beam 5A1	0.629	39 620	52 780	24.3	50.4	0.023	0.20	0.130	0.064	
Beam 5B1	.383	40 610	54 220	17.4	29.8	.052	.22	.095	.052	
Beam 5F1	.754	41 680	61 130	34.0	62.4	.085	.38	.120	.095	
Beam 5L1	1.270	38 950	66 700	49.8	51.6	.31	.59	.011	.030	
Beam 5C1	.752	41 566	59 036	28.4	60.2	.085	.38	.120	.095	
Beam 5C1	.502	38 450	54 220	23.5	38.0	.....	.....	.....	.....	

SAMPLES TAKEN FROM REPRESENTATIVE TESTS OF REINFORCING STEEL USED IN CONCRETE SHIPS

Mobile	1.125	52 130	98 050	15.0	22.7	0.47	0.70	0.04	0.04	Shell discard.
Jacksonville	.625	68 420	99 130	20.5	43.0	.44	.67	.035	.035	Do.
San Francisco	1.250	37 940	60 810	24.3	.....	.21	.44	.031	.037	Structural.
San Diego	.750	36 070	61 920	25.7	.....	.22	.44	.014	.037	Do.
Wilmington	.750	45 840	63 690	25.6	54.4	.16	.38	.046	.058	Do.
Barges	[a].50	55 760	93 130	14.0	.....	.41	.50	.02	.04	Do.

[a] Square bars.

Samples of the steel taken from beams 5A1 and 5B1 for microscopic examination showed the microstructure of wrought iron which was contaminated more or less with prominent steel streaks. They had the appearance of an inferior grade of wrought iron.

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The chemical tests showed that the phosphorous content in the reinforcement generally exceeded the limit for reinforcing bars,[4] and the percentages of manganese and carbon were much lower than those ordinarily found in structural steel. It seems evident that some of the steel obtained for these tests was of an inferior grade, although it was purchased as a good grade of structural reinforcement. Representative tests which were made in the inspection of reinforcing bars used in concrete ships and in concrete barge construction for the Inland Waterways Commission gave the results shown in Table 6. A comparison of the chemical analyses given in this table shows wide variations and indicates the importance of a careful inspection of all steel which is supplied for use in reinforecd concrete construction.

3. MAKING AND STORAGE OF TEST SPECIMENS

The beams for these tests were made at the Bureau of Standards, Washington, D. C. The concrete was mixed in the proportion of 1 part of Portland cement, two-thirds parts of sand, and 1⅓ parts of gravel, by weight, for all of the beams except 5C1, in which the mix was 1 part of Portland cement, three-fourths parts of sand, and 1½ parts of gravel, by weight. A rich mix was chosen in order to make the conditions closely similar in this respect to those in concrete ship construction.

The water used was 12 per cent, by weight, of the total dry materials. Calcium chloride was added to the water to accelerate the hardening of the concrete. Four pounds of calcium chloride were used with each 96 pounds of the mixing water.

A Snell mixer was used for mixing the concrete. Each batch was mixed for five minutes at a speed of 18 revolutions per minute, and the material was moved in wheelbarrows to the forms and shoveled into place. The forms used were made of wood. Beams 5A1 and 5B1 were puddled by means of one-half by 2 inch wooden paddles, while in the remaining beams the forms were rapped with a hand hammer during the period of placing the concrete. The top face of each test beam was troweled and then covered with a 6-inch board. The wet burlap, which was spread over all, was sprinkled daily until the forms were removed. The forms were removed after three days, and the beams were stored in moist sand at the normal temperature of the laboratory and kept moist during the period of storage.

[4] Standard Specifications for Billet-Steel Concrete Reinforcement Bars, A. S. T. M., Standards, p. 148; 1918.

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4. CONTROL SPECIMENS

Four 6 by 12 inch cylinders, made with each beam and stored under the same conditions as the beams, were used as control specimens and were tested in compression. Results of these tests may be found in Table 7. A density calculation was made for three of the beams and is recorded in the table.

TABLE 7.—Test Results for Control Cylinders

Beam No.	Proportions of mix (by weight)	Density	Date of molding, 1918	Age of beam at start of test	Control cylinders	
					Age when tested	Average compressive strength
				Days	Days	Lbs./in.²
5A1	1:⅔:1½	0.745	Apr. 24..	63	163	5596
5B1	1:⅔:1½	.745	...do....	147	163	5563
5C1	1:¾:1½	.752	May 18..	155	139	6235
5F1	1:⅔:1½	.....	June 28..	124	98	4205
5L1	1:⅔:1½	.....	Sept. 9...	71	.....	.....

5. GAGE LINES

In beams 5A1, 5B1, 5C1, and 5F1 two horizontal gage lines, 8 inches long, were located on each side of the beam at the center. These gage lines were placed in the outside bars in both top and bottom reinforcement and are designated in Figs. 17 to 26 as north top, north bottom, south top, and south bottom.

The gage holes in the reinforcing bars were generally drilled with a No. 50 drill (0.07 inch) to a depth of about $\frac{3}{32}$ inch. They were countersunk slightly with a $\frac{3}{32}$-inch drill in all cases except for beam 5B1, the only beam which had bars as small as three-eighths inch in diameter.

In beam 5L1, which had only one bar in the top and one in the bottom, the gage lines were located only on the south side at the center of the span.

6. METHOD OF TESTING

The surfaces of the beams were whitewashed in order to facilitate the detection of cracks in the concrete during the tests. After a beam had been properly placed in the testing machine, a complete set of initial readings was taken as a basis for determination of deflections and deformations. In the case of beam 5L1 initial readings were also taken of the four Ames-gage micrometers which

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were attached to the ends of the reinforcing bars for measuring the slip.

After the initial readings had been taken, the predetermined total load was applied statically in the downward direction and a new set of readings of deformation, deflection, and crack widths was taken. The cracks were traced on the beam with a pencil. The same operation was repeated with the upward load applied statically and the downward load released.

The machine was then started running and the test proceeded. In the conduct of the tests it was not possible to keep the machine running all of the time, but, when circumstances permitted, the machine was operated continuously day and night. For beams 5A1, 5F1, and 5L1 the continuity of the tests is shown in Fig. 25. For beam 5B1 the continuity is shown in Fig. 13. The vertical lines in these graphs indicate periods of rest.

At periodic intervals two readings of the strain gage were taken on each gage line, first with the upward load on the beam and then with the downward load. It was not possible to take these readings for upward and for downward load on consecutive reversals of load, but they were taken as close together as possible. Strain-gage readings were also taken at intervals with the machine not running and with no load on the test beam. When beam 5L1 was tested the slip readings for the upward load and then for the downward load were taken while the machine was in operation and as nearly simultaneously as possible.

Deflection readings were taken at intervals when the machine was in operation for the load upward and then for the load downward and generally preceding each series of steel deformation readings. Deflection readings were also taken with the machine not running and with no load on the test beam. Readings were taken on the reference beam before and after each set of deflectometer readings.

Crack widths were measured by estimation to the nearest 0.001 inch by means of a steel scale graduated to 0.01 inch and read through a glass magnifying about 4½ diameters. The crack widths were observed with the machine stopped and with the full static load on the beam. This was done for both the upward and the downward loads. Careful observations of the beam were made during the progress of the test and the cycle number at which each crack appeared was recorded on the beam just opposite the crack. As the crack increased in length the number of cycles

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was recorded for various stages of the crack. Figs. 9 and 11 are views taken after completing some of the tests and show the progress of the cracks for the beams which were tested to failure.

The cracks were all outlined by means of a heavy pencil in order to make them visible in the photographs. The test of beam 5C1 was discontinued before failure occurred, and the cracks which had developed up to the time of stopping the test are sketched in Fig. 10. Each value of crack width plotted in Figs. 12 to 16 is one-half of the sum of all of the crack widths on both sides of the beam, as measured about one-fourth inch below the top or above the bottom of the beam. Each value represents approximately the total extension either in the upper or the lower surface of the beam due to the opening of cracks.

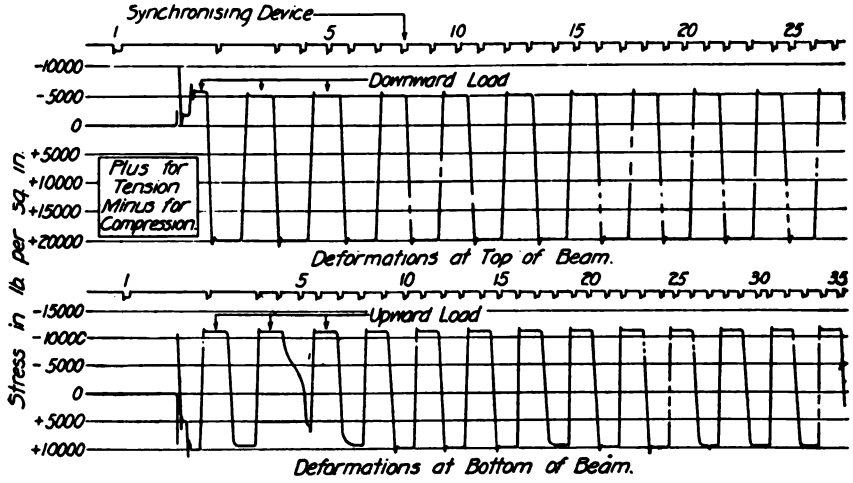

FIG. 6.—*Strainagraph record showing characteristic behavior of beams during test*

A strainagraph record,[5] taken during a portion of the test of one of the specimens, is shown in Fig. 6. This brings out some of the characteristics of the action of the testing machine.

It is found that the full load was in position about 80 per cent of the time and that about 10 per cent of the time of each cycle was required for the application of the load and 10 per cent for its removal. Assuming 17 cycles per minute as the ordinary rate of operation, it is found that the application of each load required about three-fourths of a second, and that for each 24 hours in which the machine operated the specimen was under full load about 20 hours.

[5] F. R. McMillan, The Strainagraph and its Application to Concrete Ships, Proc. Amer. Concrete Inst., 15, p. 108; 1919.

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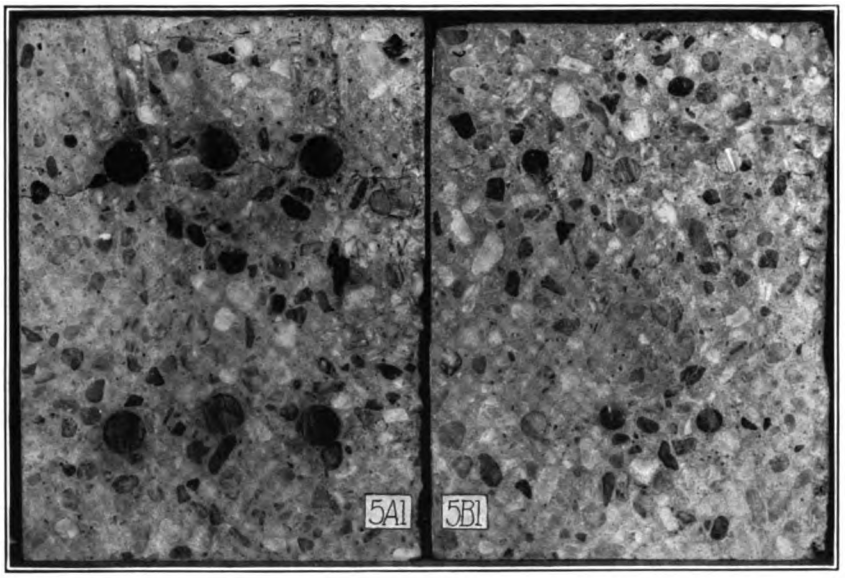

FIG. 7.—*View of cross sections through beams 5A1 and 5B1 after test*

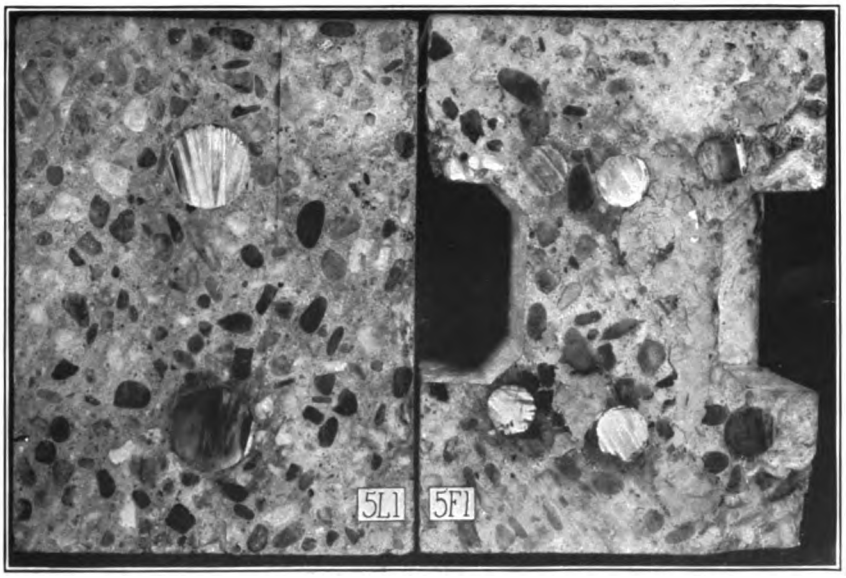

FIG. 8.—*View of cross sections through beams 5L1 and 5F1 after test*

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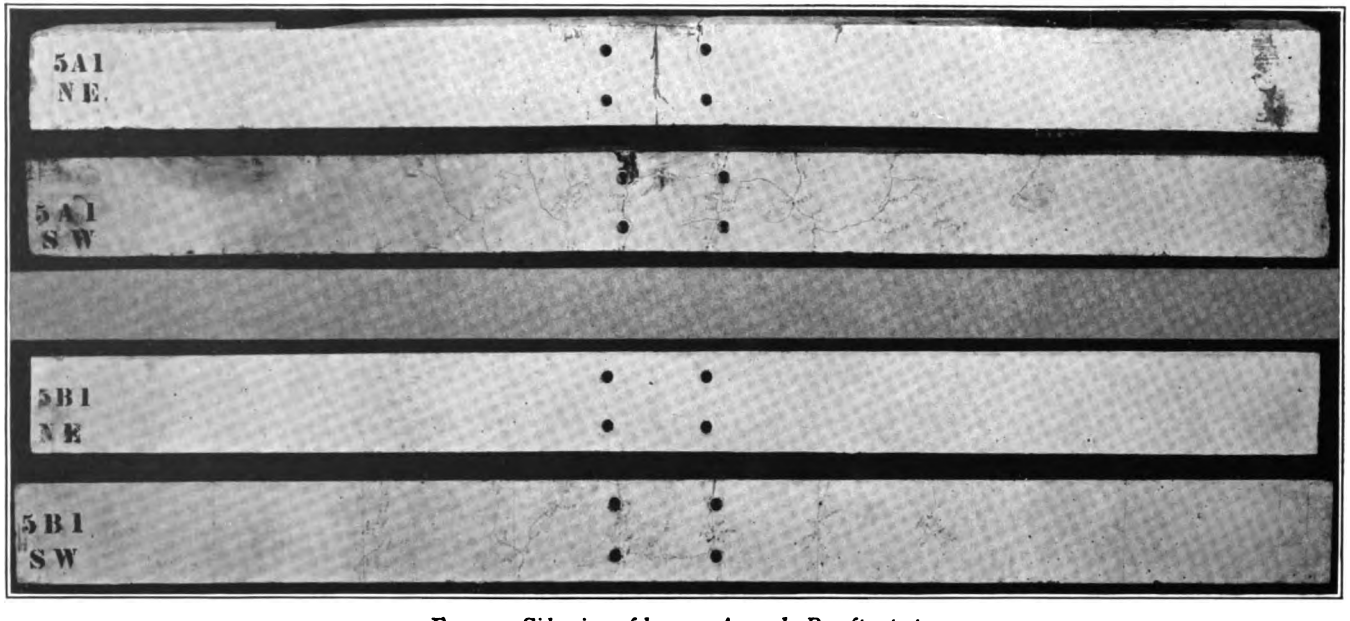

FIG. 9.—*Side view of beams 5A1 and 5B1 after test*

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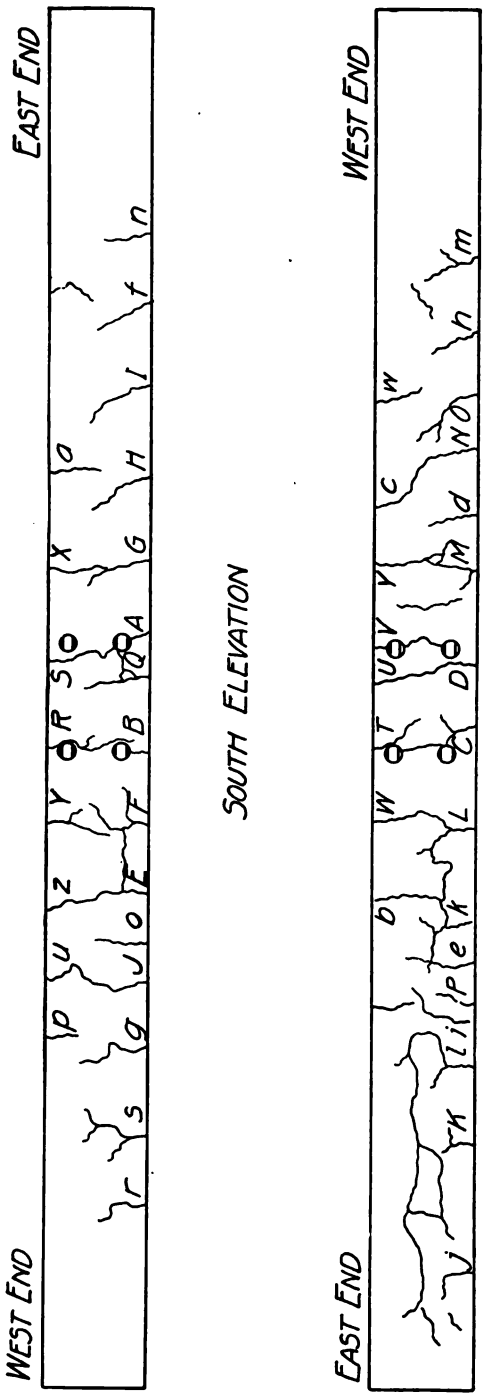

FIG. 10.—*Sketch showing location of cracks on beam 5C1 after test*

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That the stress due to impact was small is shown by the form of the graph. The only portion of the graph in which anything like an effect of impact appears is the peak which occurs with each application of the load. The height of this peak above the horizontal portion of the graph is about 4 per cent of the total height of the graph. Assuming that the peak is caused entirely by impact on the beam, the stress due to impact would be about 4 per cent of the sum of the tensile and compressive stresses due to static load. Since the inertia of the mechanism of the strainagraph may have been partly responsible for the occurrence of the peak, it is possible that the impact may have been smaller than the amount stated, but it does not seem that it can have been larger than that.

IV. TEST DATA AND DISCUSSION

1. TABLES AND DIAGRAMS

The measured dimensions and computed properties of the test beams are given in Table 8.

TABLE 8.—Measured Dimensions and Computed Properties of the Test Beams

Dimension	Notation	Beam numbers				
		5A1	5B1	5C1	5F1	5L1
Nominal cross section	.....	6 x 8 in.	6 x 8 in.	6 x 8 in.	I section	6 x 8 in.
Width of beam, inches	b	6.03	6.00	6.00	.....	6.00
Distance, compression face to center of tension steel, inches	d	6.00	5.93	6.00	6.00	6.00
Distance compression face to center of compression steel, inches	d′	2.05	2.07	2.00	2.00	2.00
Sectional area bottom steel, square inches	A_s	.9204	.3312	1.325	1.325	1.227
Sectional area top steel, square inches	A'_s	.9204	.3312	.588	1.325	1.227
Ratio A_s/bd	p	.0254	.0093	.0368	.....	.0341
Ratio A'_s/bd	p′	.0254	.0093	.0163	.....	.0341
Ratio of depth of neutral axis to distance compression face to center of tension steel	k	.461	.348	[a].530 [b].393	.495	.490
Ratio moment arm of resisting couple to depth *d*	j	.808	.884	[a].794 [b].823	.....	.784
Distance between center of compression area and center of tension steel, inches	j1	4.848	5.242	[a] 4.764 [b] 4.938	4.76	4.704

[a] Downward. [b] Upward.

NOTE.—In these computations n is taken as 10.

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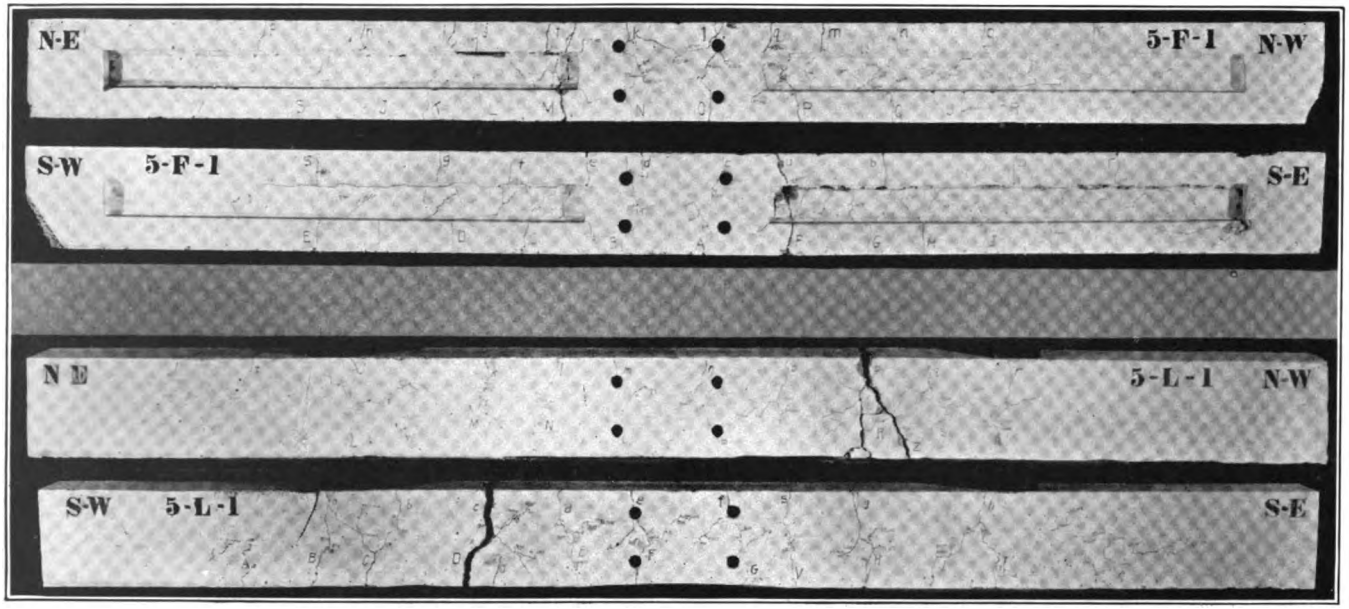

FIG. 11.—*Side view of beams 5F1 and 5L1 after test*

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General results for the beams tested are given in Table 9. The values of the measured tension and measured compression in the reinforcement given in this table are determined directly from the unit deformations given in Figs. 17 to 23. These values for each beam represent the stresses at a time when the beam had reached approximately a stationary condition of stress. The compressive stresses given were based on the assumption that the total compressive stress in the concrete was equal to the numerical difference between the total tension and the total compression in the reinforcement of the beam. The stresses were assumed to be dis-

TABLE 9.—Results of Reversal-of-Stress Tests

Beam No.	Reinforcement					Measured tension in steel f'_s		Measured compression in steel f'_s	
	Top		Bottom		Bar ends	Top, load upward	Bottom, load downward	Top, load downward	Bottom, load upward
	Number of bars	Diameter	Number of bars	Diameter					
		Ins.		Ins.		Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²
5A1	3	5/8	3	5/8	Hooked	21 600	19 500	5400	7000
5B1	3	3/8	3	3/8	...do	21 900	22 800	.....	.....
5C1	3	1/2	3	3/4	...do	1 500	11 000	7500	.....
5F1[a]	3	3/4	3	3/4	...do	11 400	11 700	6900	7000
5L1	1	1 1/4	1	1 1/4	Straight	18 000	17 400	4000	4800

Beam No.	Computed compression in concrete f_c		Range of stress in steel from tension to compression		Number of cycles at failure	Remarks
	Top, load downward	Bottom, load upward	Top	Bottom		
	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²		
5A1	1565	1555	27 000	26 500	709 041	Three top bars failed in tension at east gage point.
5B1	1210	1160	21 900	22 800	59 377	Three top bars and one bottom bar failed in tension at east gage point.
5C1	157	1435	9 000	11 000	.....	Test discontinued at 2 008 000 cycles without sign of failure.
5F1	.....	.....	18 300	18 700	544 448	Top middle bar, all bottom bars, and the web steel failed in tension.
5L1	2080	2020	22 060	22 200	431 821	Both bars failed in tension 16 inches west of center.

[a] Web reinforced with one-quarter inch wire loops, 2-inch pitch: I section.

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tributed according to a straight-line relation. The equation which gives these stresses is

$$f_c = \frac{2(f_s A_s - f'_s A_s)}{b(xy + d') - A_c}, \text{ in which}$$

f_c = extreme fiber compressive stress in concrete,
f_s = observed tensile stress in reinforcement,
f'_s = observed compressive stress in reinforcement,
A_s = area of reinforcement in tension,
A_c = area of reinforcement in compression,
b = breadth of beam,
y = depth from center of compression reinforcement to center of tension reinforcement,
d' = distance from compression surface to center of compression reinforcement,
$x = \frac{f'_s}{f'_s + f_s}$ = ratio of distance between neutral axis and compression reinforcement to distance between tension and compression reinforcement.

The summations of the crack widths which were measured at intervals during the tests, at levels one-fourth inch above the bottom and one-fourth inch below the top of the beams, are shown in Figs. 12 to 16. These figures show also the measured deflections. Letters were used in alphabetical order to identify the cracks as soon as they were discovered. These letters may be distinguished in Figs. 9 to 11. Figs. 17 to 23 show the number of cycles plotted as abscissas and the unit deformations as ordinates.

2. BEHAVIOR OF BEAMS DURING TESTS

In the following paragraphs are given the more important phenomena of the tests.

(*a*) Beam 5A1 had three ⅝-inch plain round bars in the top and three in the bottom. The applied load was 5000 pounds both upward and downward. This beam was tested to show the effect of repeating a compressive stress in the concrete of about 2000 pounds per square inch, combined with a relatively large range of stress from tension to compression in the steel. A stress of 1500 pounds per square inch was used in the design of the concrete ships.

Fig. 12 shows the relation between the deflection either upward or downward and the total crack widths in the top and in the bottom of the beam throughout the test. Figs. 17 and 18 show

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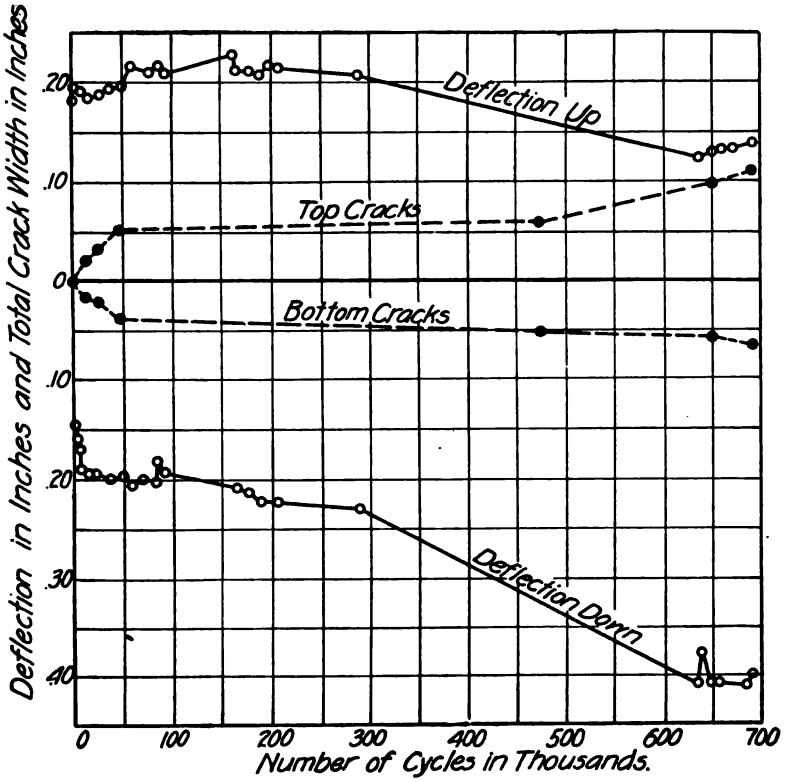

FIG. 12.—*Deflection and total crack widths for beam 5A1*

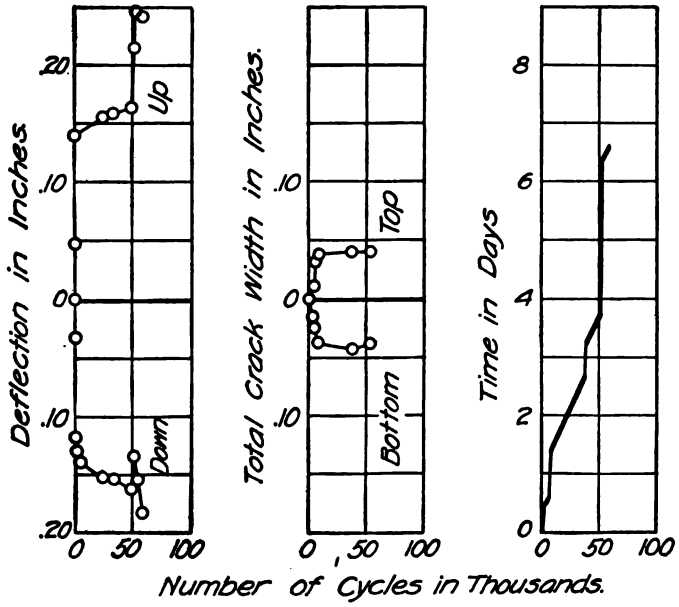

FIG. 13.—*Deflection crack widths and continuity of test for beam 5B1*

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the unit deformations in the reinforcing steel as ordinates and the number of cycles of the machine as abscissas.

During the first 1000 cycles[6] the widths of the tension cracks increased to a total of 0.025 inch in the top and 0.020 inch in the bottom of the beam (Fig. 12). As the tension cracks increased in number and size the steel deformations changed slightly with a marked increase in deflection of the beam both upward and downward. After the first 1000 cycles the crack widths, deflections, and deformations remained almost constant through about

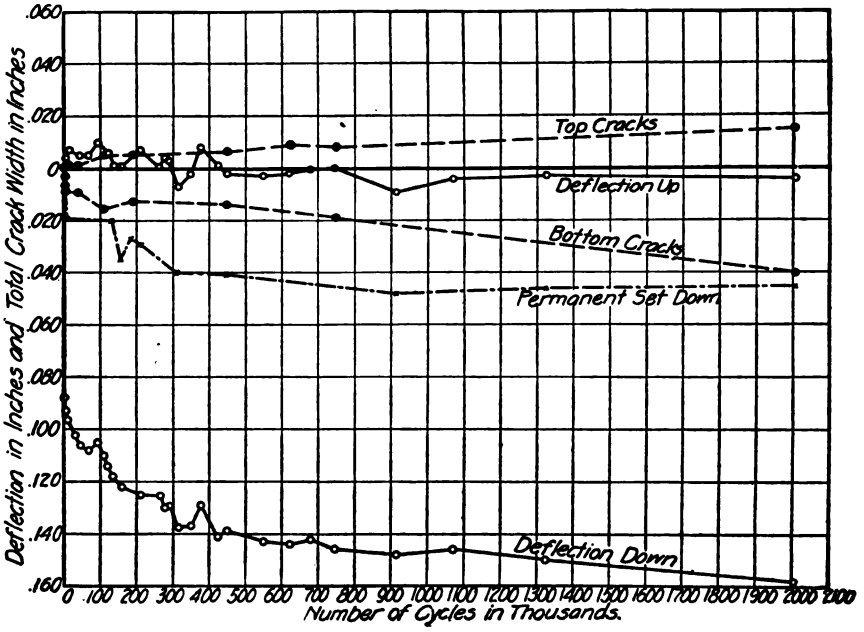

FIG. 14.—*Deflection and crack widths for beam 5C1*

one-half the duration of the test, when the deflection downward and the crack widths increased gradually until the beam failed.

The cracks increased in length, and toward the end of the test many of them extended from the top of the beam to the bottom. The cracks in the top of the beam did not close entirely with downward load, due, apparently, to some small particles of concrete chipping off and falling into the cracks. As a result the cracks in the top of the beam appeared somewhat ragged, but in the bottom of the beam they were still clean cut. The top steel failed in tension at 709 041 cycles at the west gage point, which was 4 inches west of the center of the span.

[6] An error has been found in Fig. 12. For all plotted points in this figure showing crack widths the number of cycles should be reduced by 13 500.

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(*b*) Beam 5B1 had three ⅜-inch plain round bars in the top and three in the bottom. This beam was designed so that when the load was applied the computed tensile stress in the steel would be high (28 800 pounds per square inch), and the computed

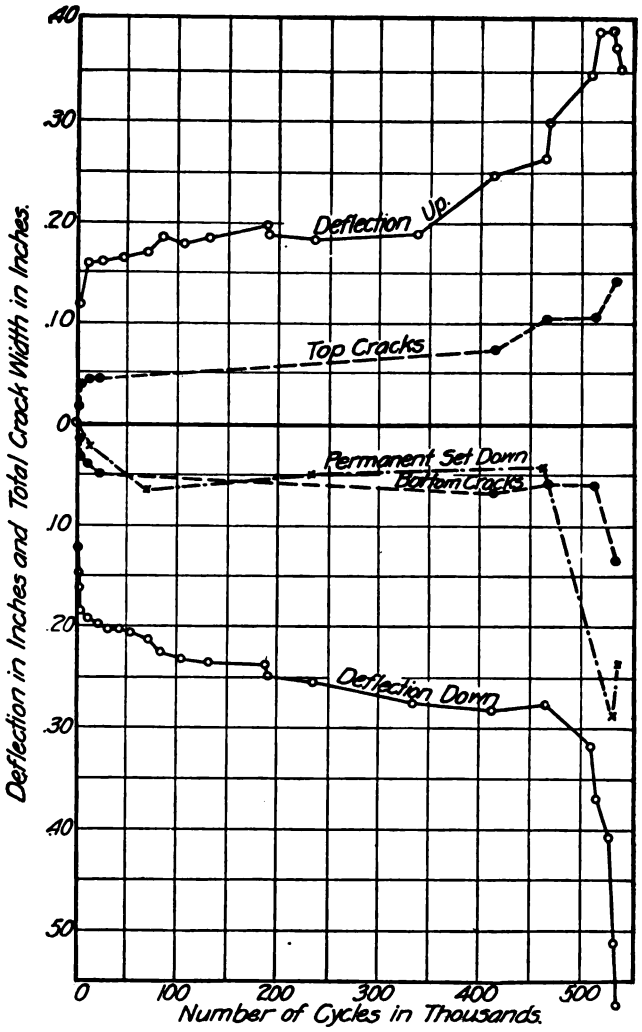

FIG. 15.—*Deflection and crack widths for beam 5F1*

compressive stress in the concrete would be about equal to the working stress used in the design of the concrete ships. The applied load was 2380 pounds both upward and downward. The deflections, crack widths, and the continuity of the test are shown in Fig. 13. The deformations are shown in Fig. 19.

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No cracks appeared when a static load of 2380 pounds was first applied, but cracks *A* to *P*, Fig. 9, occurred on the first repetition of the load. During the first 300 cycles the deflections increased rapidly, while the crack widths increased rapidly during the first 7000 cycles. Both deflections and crack widths then remained

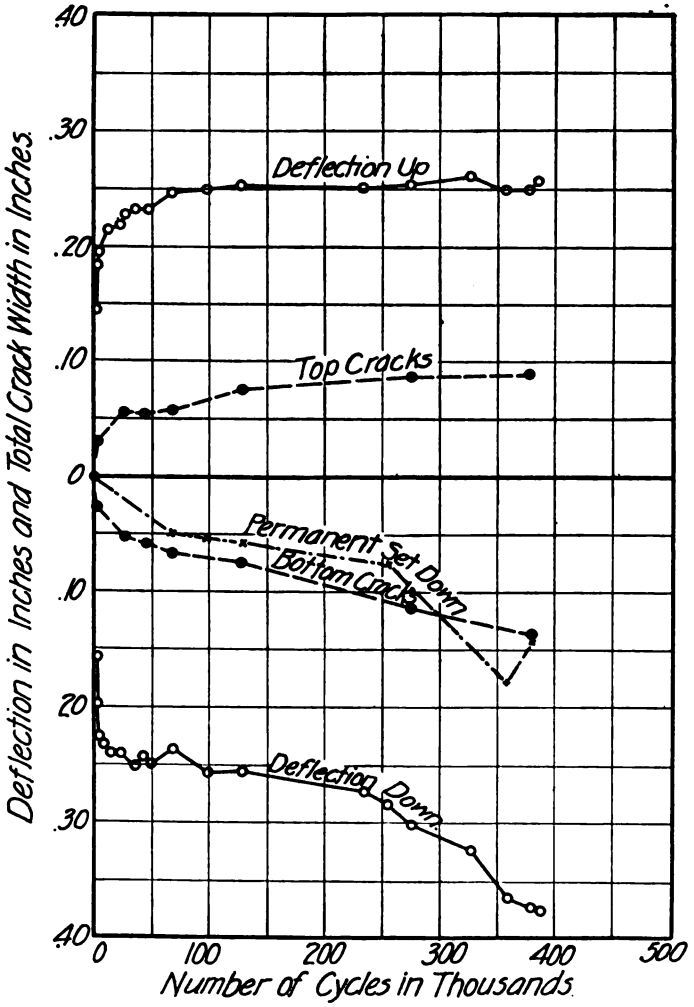

FIG. 16.—*Deflection and crack widths for beam 5L1*

almost stationary until the beam was close to failure. The deformations show less regularity than the deflections and crack widths throughout this portion of the test. In the last measurements taken before failure the deflections and deformations generally showed a great increase, but the crack widths did not.

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At 59 377 cycles all three of the top bars broke without any necking which could be detected by visual observation. The south bar broke in the east gage hole, the north bar in the west gage hole, and the middle bar at a point opposite the west gage hole.

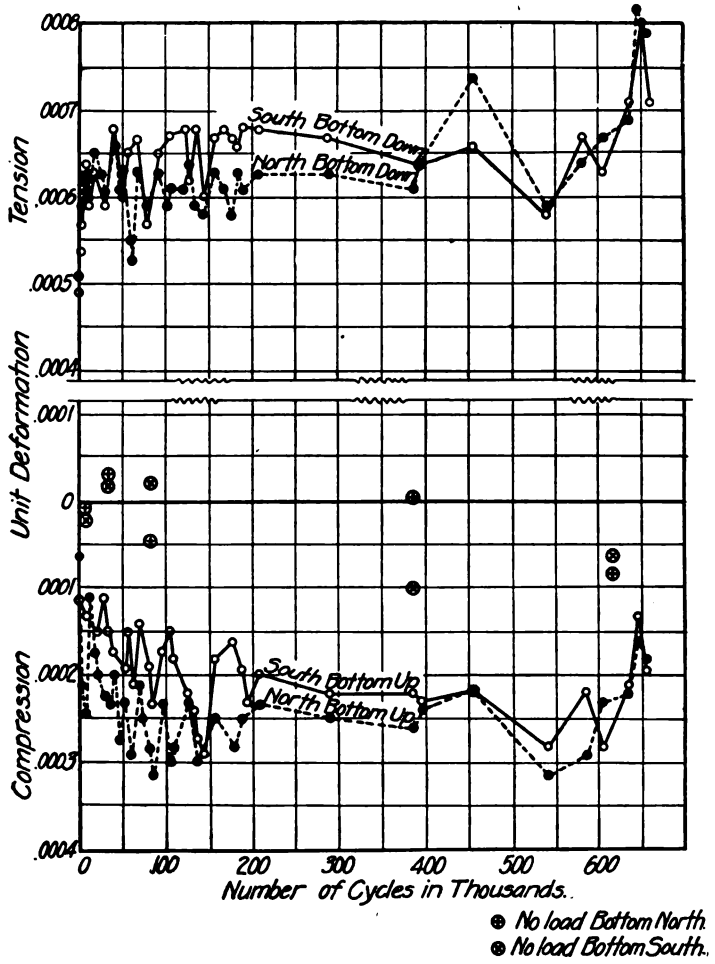

FIG. 17.—*Unit deformations in bottom of beam 5A1*

(*c*) Beam 5C1 had three ¾-inch plain round bars in the bottom of the beam and three ½-inch plain round bars in the top. This beam was designed so that the test load would produce stresses not greater either in tension or compression than the working stresses used in the concrete ship design. The applied load was 3630 pounds in the downward direction and 670 pounds in the

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upward direction. Fig. 14 shows the crack widths and deflections throughout the test. Fig. 20 shows the unit deformations as the test progressed.

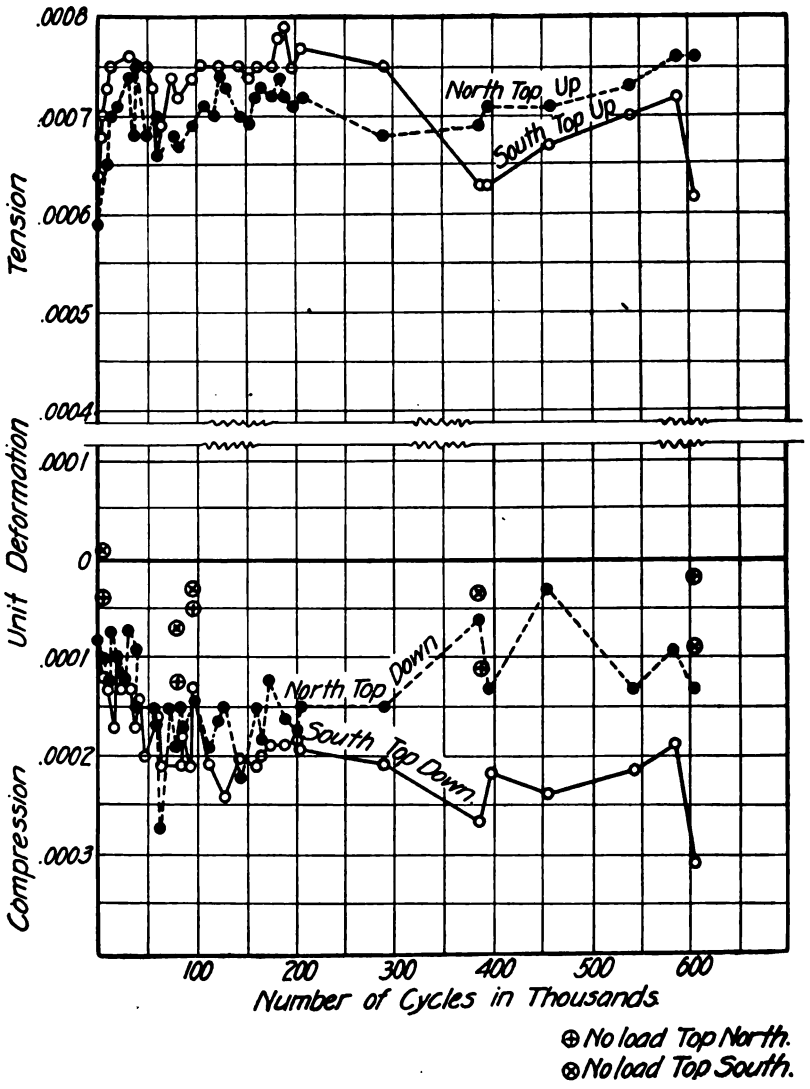

FIG. 18.—*Unit deformations in top of beam 5A1*

The upward deflections and the crack widths in the top of the beam increased slightly at the start but remained almost constant throughout the remainder of the test. The downward deflection, the downward permanent set, and the crack widths at the bottom of the beam increased gradually through about 400 000 cycles

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of load. The test was discontinued at 2 008 000 cycles of load and had shown very little change after passing 400 000 cycles. The cracks were very small and extended only a short distance from the top and bottom surfaces of the beam. No serious

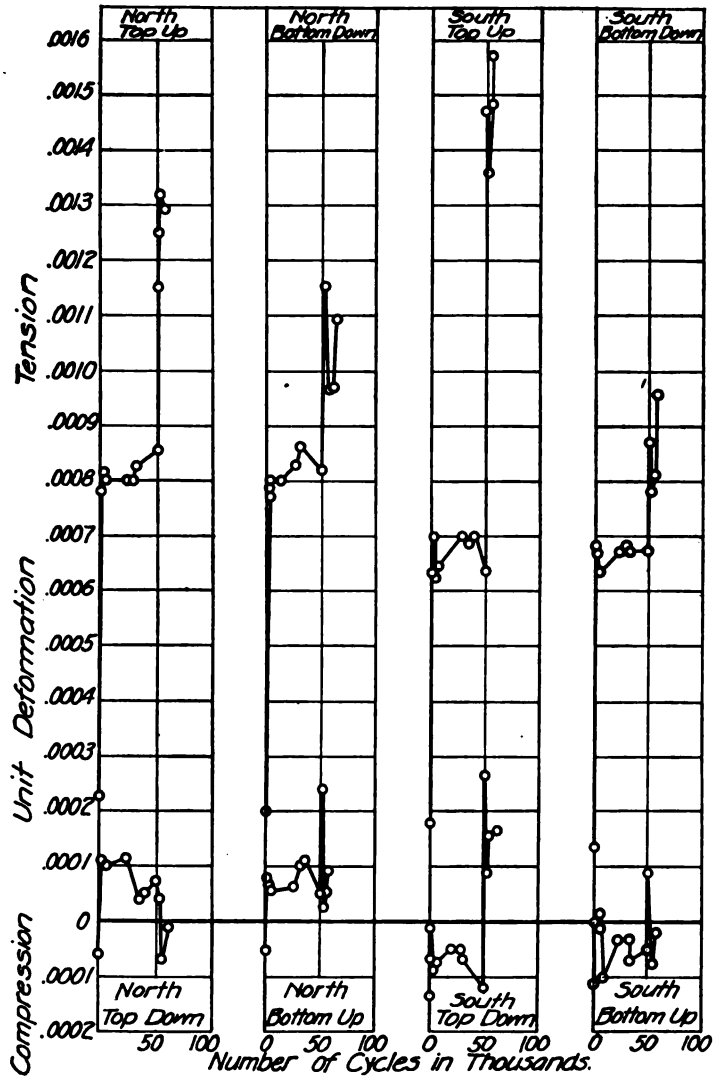

FIG. 19.—*Unit deformations in beam 5B1*

developments of any kind showed indications of approaching failure.

(*d*) Beam 5F1 was made with an I-shaped cross section with three ¾-inch plain round bars in the top flange and three in the

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bottom flange, with web reinforcement consisting of a ¼-inch plain round bar wound around and welded to the top and bottom central bars with a pitch of 2 inches. This test was designed to develop high shearing stresses in the web without affecting appreciably the resistance to tensile and compressive stresses. The applied load was 5000 pounds both upward and downward. This gave a computed shearing stress of 175 pounds per square inch. Fig. 15 shows the deflections, crack widths, and permanent set downward as the test progressed. Figs. 21 and 22 show the unit deformations.

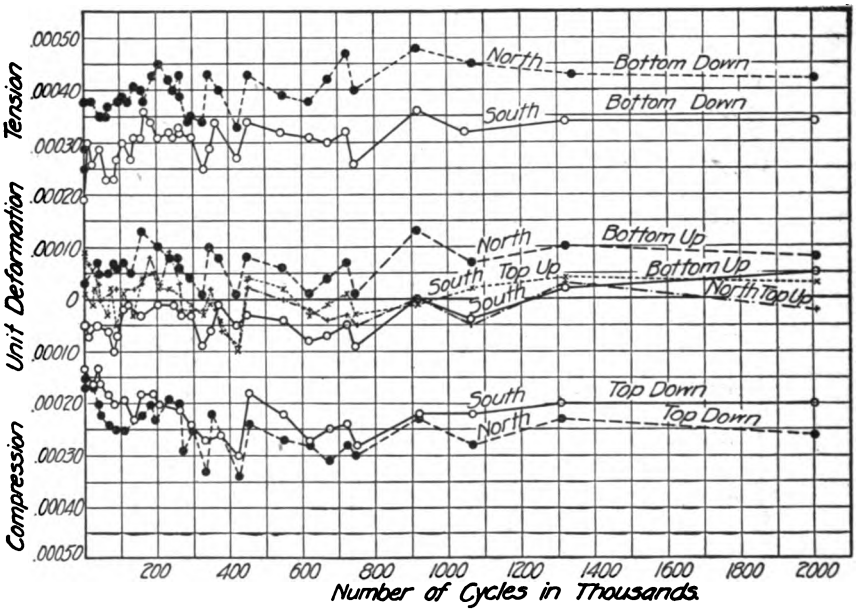

FIG. 20.—*Unit deformations in beam 5C1*

The deflections in both directions of loading increased rapidly with a gradual increase in crack widths during the early stages of the test. At 3600 cycles a horizontal crack appeared at the top of the web extending the whole length of the I-shaped portion east of the center of the beam. From the horizontal crack small diagonal cracks extended downward toward the center of the web as the test continued. Similar cracks appeared later at the junction of the web with the lower flange. From 3600 cycles until the test was about three-fourths completed, the crack widths and deflections remained almost constant but the unit deformations varied considerably.

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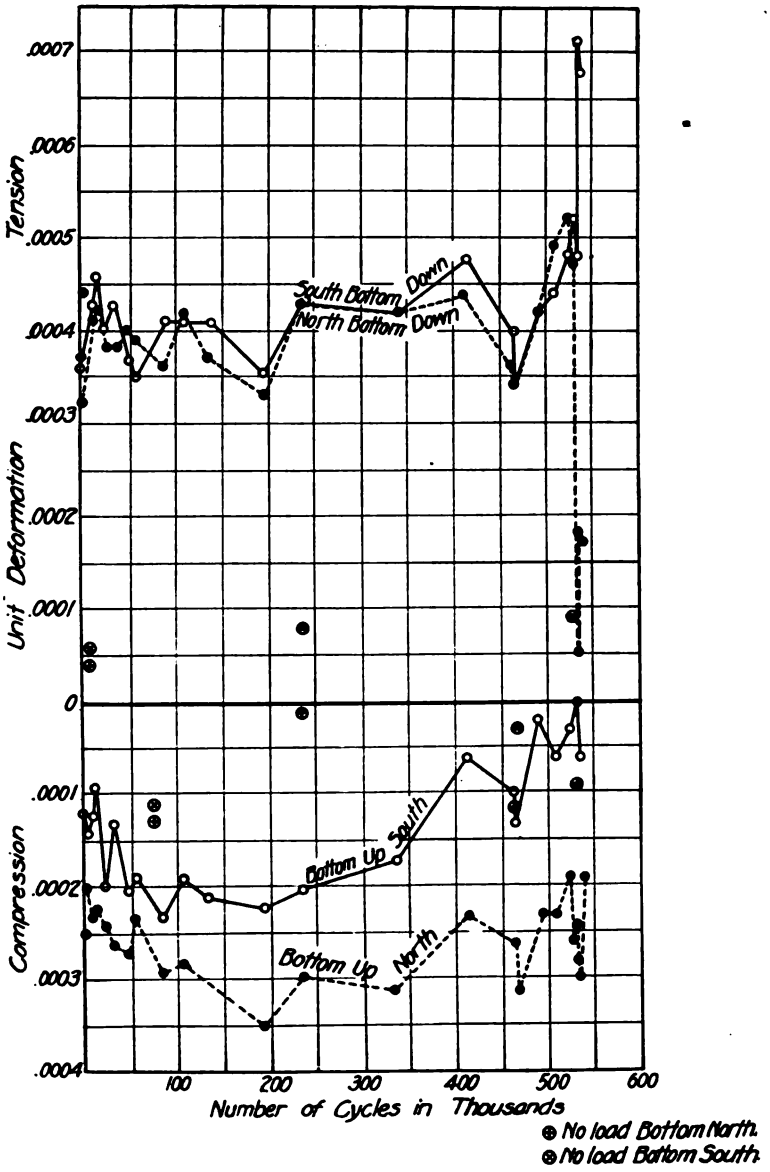

FIG. 21.—*Unit deformations in bottom of beam 5F1*

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At 467 000 cycles the cracks in the top and bottom flanges and the vertical web cracks just east of the east load point, at the section where the web thickness was reduced, showed a marked increase in size and failure seemed imminent at this point. Rust and pulverized concrete were working out of the cracks at this

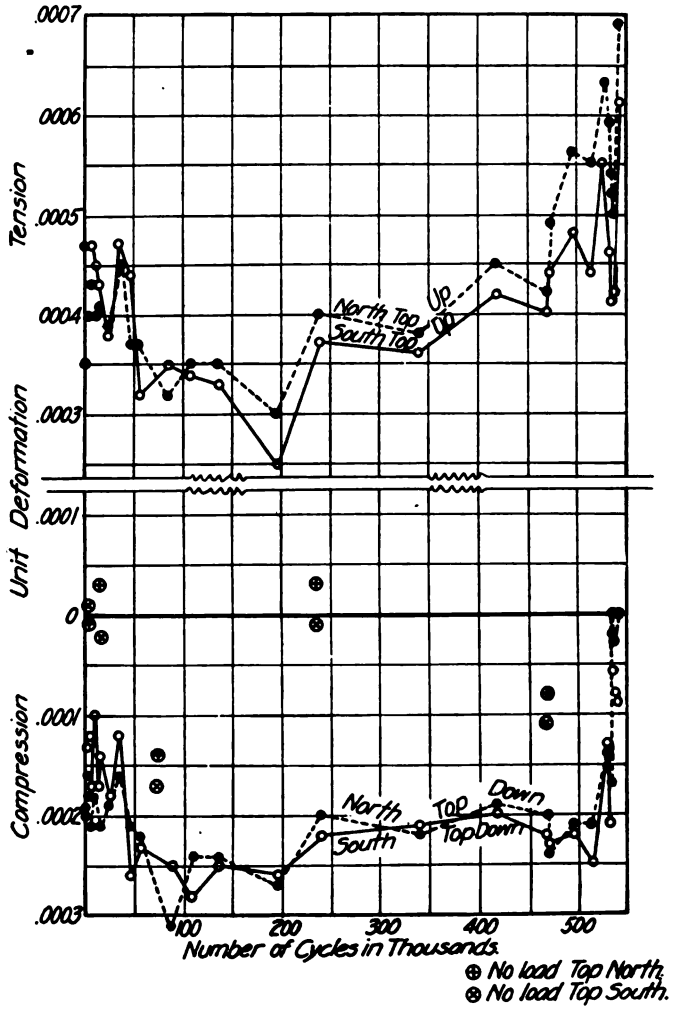

FIG. 22.—*Unit deformations in top of beam 5F1*

point and along the horizontal crack under the top flange of the beam. Apparently the horizontal crack passed under the horizontal bars and separated the top flange from the web in the east portion of the beam. A horizontal movement of the flange longitudinally with reference to the web was observed. This

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movement was 0.015 inch when first measured. As the separation of flange and web became more nearly complete the movement increased, until shortly before failure it was as much as 0.07 inch.

The total crack width just before failure was 0.141 inch at the top surface and 0.133 inch at the bottom surface of the beam. The steel bars became visible in several places where the concrete was chipping off and considerable slipping of the bars in the top was noticeable with the sliding of the top flange. There was little cracking of the concrete in the west portion of the beam.

During the last 100 000 cycles of the test the deflections and permanent set downward increased very rapidly, with an increase in the deformations in the reinforcing steel. At 544 000 cycles the top and bottom middle bars and both outside bars in the bottom failed in tension at points about 5 inches east of a section through the east gage holes. The web reinforcement in that section also failed in tension at the point where it was welded to the top bar. Although the end of the test was brought about by complete tension failure in the reinforcement, the separation of the flanges from the web was so extensive that, toward the end of the test, the beam sagged greatly and some of the load probably was carried by the lever which was not applying the load.

(*e*) Beam 5L1 had one 1¼-inch plain, round, straight bar in the top and another in the bottom. The beam was designed for the purpose of studying the effect on the end slip caused by repeated application of high bond stresses. The applied load was 6000 pounds both upward and downward. This gave a computed bond stress of 161 pounds per square inch. The permanent set downward, the crack widths and the deflections are shown in Fig. 16. The deformations in the steel are shown in Fig. 23. Fig. 24 shows the end slip of the bars.

A gradual increase in deflections, crack widths, and permanent set downward took place until 70 000 cycles were recorded. The top crack widths and upward deflection remained almost constant until the completion of the test. The permanent set downward, the downward deflection, and the bottom crack widths increased gradually throughout the remainder of the test.

Up to 7200 cycles very gradual movements of the ends of the bars from their initial positions were detected by the gages which were used to measure slip, but no reversal in direction of movement with the reversal of load could be detected. From this time on

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throughout the test there was a rapid increase in the amount of slip at the west end of the bottom bar and a reversal in the direction of the movement of the end of the bar with the reversal in the direction of load. It was expected that failure would occur on account of this slip, but it actually occurred after 431 821 cycles of load by tension in both reinforcing bars at a section 16 inches from the center of the span.

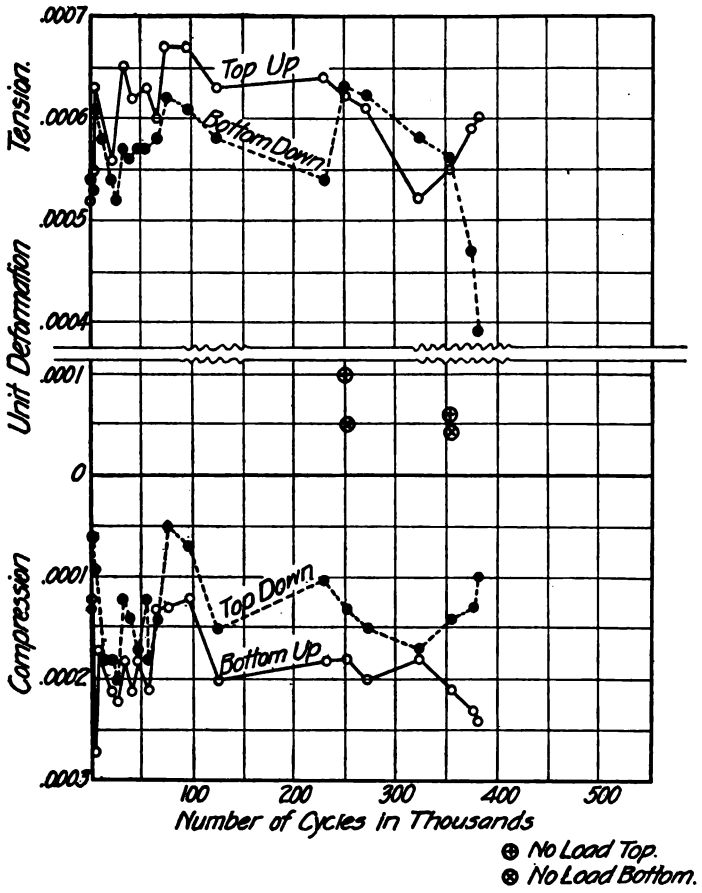

FIG. 23.—*Unit deformations in beam 5L1*

3. SIGNIFICANCE OF TENSION FAILURES

The failures of all of the beams tested have been by tension in the steel. The failures generally occurred with fewer repetitions of load than would be expected if the results of previous tests [7]

[7] Moore and Seely, The Failure of Materials under Repeated Stress, Proc. Am. Soc. Test. Mats., 15, p. 437, 1915; also Constants and Diagrams for Repeated Stress Calculations, 16, p. 471, 1916.

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be taken as a guide. It seems probable that the use of defective steel as reinforcement was partly responsible for this result. Yet it should be noted that the steel was purchased as a commercial grade of reinforcement and probably was no worse than steel that is frequently used on small jobs where little or no inspection is exercised. All of the tension failures of the steel occurred without necking of the bars. This is characteristic of failure under repeated tensile stress and, whatever may have been the grade of the steel used, the effect of the repetition of loads seems clearly to have been present.

Besides the tension failures in the steel, other effects of the loading were apparent, and if a better grade of steel had been used, it seems probable that in some cases failure would have been due to other causes than tension in the reinforcement. Thus, in beam 5A1 a ragged appearance at the edges of the cracks was noted, and if the steel had not failed when it did, critical conditions in the concrete might have developed. However, the indications at the time of failure were that the concrete was still carrying its full proportion of the compression.

In beam 5F1 the web was badly cracked before the tension failure occurred. The flanges of the beam had been almost detached from the web at one end and the deflections had become excessive. It is possible that the tension failure may have been hastened by a repeated bending of one of the reinforcing bars. At any rate, it seems likely that a failure would have developed in the web before a great length of time, if failure of the steel in tension had not intervened.

In beam 5L1 the end slip of one of the bars had increased to 0.06 inch before tension failure took place. This is well beyond the amount of slip which, in the analysis of test data,[8] has previously been recognized as critical, and it seems certain that a complete failure in bond was impending and would have been the primary cause of failure if the tension failure in the steel could have been delayed for a short time.

4. END SLIP OF BARS

Slip of bars was measured in beam 5L1 only. In all other beams the bars were hooked to prevent end slip. Fig. 24 shows the movement of the ends of the bars plotted as ordinates, and the number of cycles of load as abscissas. Movement toward the

[8] D. A. Abrams, Tests of bond Between Concrete and Steel, Bull. No. 71, Eng. Exp. Sta., Univ., of Ill., 1913.

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center of the span is plotted upward and movement away from the center is plotted downward.

Most of the slip occurred at the west end of the bottom bar. Up to 7200 cycles of load a movement of 0.0009 inch at this point

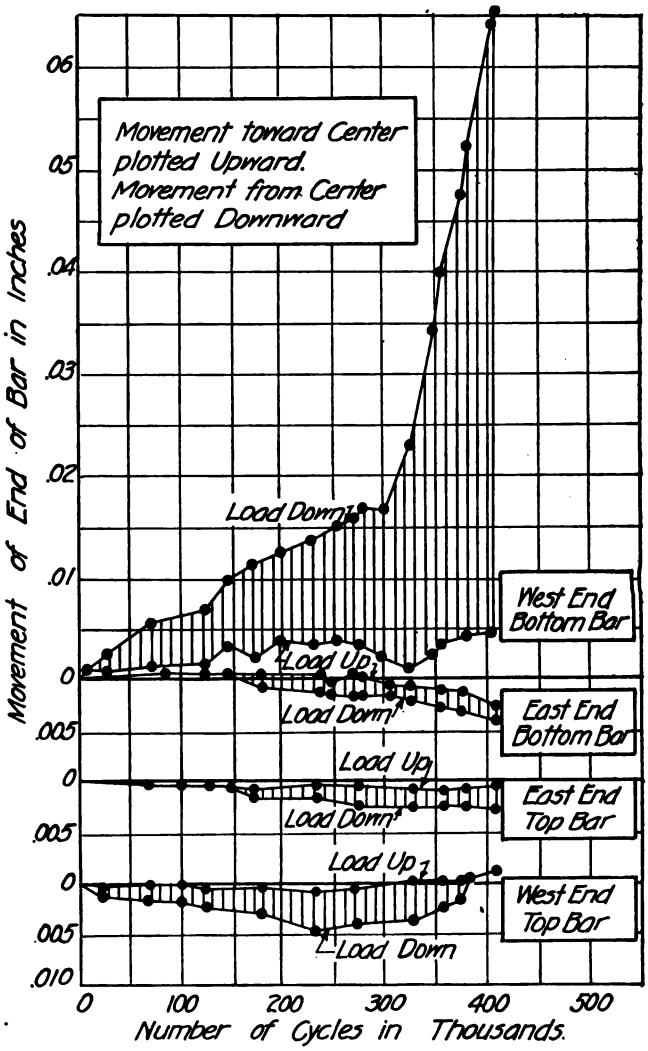

FIG. 24.—*End slip of reinforcing bars for beam 5L1*

was detected. This may have represented an actual movement of the bar with reference to its initial position, but up to this stage of the test no movement of the pointer of the Ames dial could be detected between successive applications of the load. From this time on, however, there was an instantaneous movement of the west end

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of the bar toward the center of the span when the load was applied downward, and a return to approximately the original position when the load was applied upward. This cycle of slip was traversed with each cycle of load, and as the bar reached the end of its slip the suddenness of its stopping was marked by a sharp sound as of the impact of steel against concrete. The movement toward the center of the span increased with an increase in the number of applications of load, but as the test progressed the end of the bar fell short more and more of coming back to its original position with the application of upward load.

It is a matter of interest to note that the data indicate a simultaneous movement of one end of the bottom bar toward the center of the span and of the other end away from it; that is, the entire bar would appear to have been moving through the beam from west to east. The total movement of the bottom bar was slight, however, and it is doubtful if the data afford any satisfactory explanation of this phenomenon. It is just as inexplicable that both ends of the top bar appear to have been moving away from the center of the beam as that the bottom bar, as a whole, was moving through the beam from west to east. However, the movements observed at the ends of the top bar were so small that they are unimportant in comparison with the large amount of slip which occurred at the west end of the bottom bar.

The sharp increase of slip which occurred at the west end of the bottom bar at 300 000 cycles of load seemed to forecast that failure would be due to the slipping at that position; but, after 431 821 cycles of load, failure occurred by tension in both of the reinforcing bars at a section 16 inches west of the center of the span. When the beam was broken up an examination showed that the adhesion between the lower bar and the concrete in the west portion of the beam had been entirely destroyed. Near the section of tension failure scratches on the reinforcing bar were present, which indicated an abrading action of the aggregate on the bar, due to its movement back and forth through the concrete.

It is of importance that, although up to 7200 cycles of load there was no measurable slip between successive applications of load, a continuation of the test finally produced a slip of 0.06 inch. The slip probably would have progressed to the point of causing failure of the beam if tension failure in the reinforcement had not occurred before that stage was reached.

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5. EFFECT OF GAGE HOLES ON LOCATION OF POINT OF FAILURE

As noted in Part III, article 5, the gage holes in the reinforcing bars were 0.07 inch in diameter and about three thirty-seconds inch deep. In most cases also the holes were countersunk slightly. The proportional reduction of area of the cross section of the bar due to the presence of the gage holes was comparatively small, even in the case of the smallest bars used, which were three-eighths inch round, but the effect of the gage holes in hastening failure may have been important. It has been pointed out [9] that "a sudden change of section is a great source of weakness in withstanding repeated stresses." The presence of gage holes furnished a sudden though slight change in section.

The smallest bars used would be expected to show the largest effect from the presence of gage holes. In beams 5A1 and 5B1, having five-eighths and three-eighths inch bars, respectively, failure occurred through the gage holes. In these beams the middle bars had no gage holes, but in each case failure of the middle bar occurred at a point just opposite a gage hole in the outer bar.

In all of the beams there seemed to have been a tendency for failure to occur in all of the bars at the same section through the beam. For the beams having three-eighths and five-eighths inch bars, the location of the section of failure appeared to be influenced by the location of gage holes. For the beams having three-fourths and 1¼ inch bars, the location of the section of failure was not determined by the location of the gage holes.

The number of repetitions required to cause failure was smallest for the beam 5B1, in which the proportional effect of the gage holes was largest. This seems to be in keeping with the discussion in the previous paragraphs, but the tensile stresses developed in this beam, as shown by the deformations in Fig. 19, were enough larger than those in the other beams to raise some doubt as to the correctness of the conclusion that the gage holes had an important effect in hastening failure or determining where it should occur.

If the gage holes in the five-eighths-inch bars were important in causing failure, beam 5A1 should have been expected to fail under a smaller number of applications of load than beams 5L1 and 5F1, in which failure did not occur through the gage holes. On the contrary, beam 5A1 withstood a larger number of repetitions than either 5L1 or 5F1, although the stresses developed in beam 5A1 were higher than the stresses in either of the other beams.

[9] Johnson's "Materials of Construction," fifth ed., p. 776.

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Although there are indications that the gage holes in the bars had some influence in determining the location of the section of failure, it does not seem that they had much effect upon the ultimate strength of the steel under repeated loading.

6. EFFECT OF CRACK DEVELOPMENT ON LOCATION OF POINT OF FAILURE

Even where gage holes in the bars appeared to determine the location of the section of failure, there was also a marked development of cracks at the same section. In fact, in all cases tension failure of the reinforcing bars occurred at sections where large cracks extended entirely through the section of the beam.

At the sections where tension failure occurred the maximum widths of the cracks just before failure were as given in Table 10. These were the largest cracks in the beams. In beams 5A1 and 5L1 it was noted that the cracks did not seem to close entirely with each reversal of load. If the fractured surfaces were abraded to such an extent as to render ineffective some of the concrete, the reinforcement would have been forced to take more than its share of the compressive stresses and that would have increased the range of stress in the steel, traversed in each cycle, above that given in computations of stress (Table 1) based upon the unity of the entire section. On the contrary, the data taken as the test progressed do not show any more rapid increase in compressive stress than in the tensile stress, and this would discredit an explanation of failure based upon an assumed increase in compression in the steel as the test progressed.

TABLE 10.—Width of Cracks at Points of Failure

Beam No.	Designation of crack measured	Measured width of crack	Number of cycles at time of measurement	Number of cycles at failure
		Inch		
5A1	B	0.012	648 241	660 000
	F	.025		
	H	.023		
5B1	A	.000	57 387	59 377
	F	.000		
	J	.013		
	N	.002		
5F1	u	.025	532 490	544 448
	F	.012		
	M	.023		
5L1	D	.070	377 450	431 821
	R	.008		
	Z	.070		
	c	.040		
	p	.040		

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If the reinforcement had taken all of the compression, the stresses in tension and in compression should have been equal. The fact that the tensile stresses were much larger than the compressive stresses indicates that the concrete must have assisted greatly in resisting the compressive stresses. Although the cracks seemed not to be entirely closed, this must have been merely a surface condition.

At the sections where failure finally occurred there developed, toward the end of the tests, a change in slope of the beams so sharp that it could be observed by visual inspection without instruments. This phenomenon, whatever be its cause, would be expected to introduce a sharper curvature of the bar at this point than the elastic curvature of the beam as a whole. Bending stresses in the reinforcing bars independent of those in the beam as a unit would result, and this may assist in explaining the early failure of some of the beams.

7. EFFECT OF REPETITION OF BENDING STRESSES ON PHYSICAL PROPERTIES OF STEEL REINFORCEMENT

The physical tests indicated very little change in the properties of the steel due to the repetition of load on the beam. Table 11 shows a comparison of yield point, ultimate strength, ultimate elongation, and reduction of area for specimens of the steel taken before making the beams and for specimens taken from the beams after failure. Each value is derived from the tests of four to nine specimens. This table shows that the original values were slightly smaller than those obtained from specimens cut from three of the beams after the reversal-of-stress tests, but the original values were somewhat greater for the specimens applying to beam 5B1 than the average values obtained from specimens cut from the beam after failure. In most cases the difference is so slight that it is probably accidental.

TABLE 11.—Comparison of Properties of Reinforcing Steel after Repeated Loading with Properties before Repeated Loading

[Values are stated in percentage in corresponding values obtained before use in beam tests]

Beam No.	Yield point	Ultimate strength	Ultimate elongation	Reduction of area
	Per cent	Per cent	Per cent	Per cent
5A1	106.0	102.3	132.9	112.3
5B1	99.0	104.8	98.3	95.5
5F1	100.0	103.4	119.8	103.6
5L1	103.0	100.6	101.7	103.6

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In the failures under repeated stress, no necking of the bars could be detected, and this is characteristic of fatigue tests of steel. Almost the same ultimate elongation was shown in the tension tests of the steel specimens taken from the bars before molding of the test beams and from the bars which had been subjected to repeated stress in the beams. This indicates that the general structure of the steel was not affected by the repeated reversal of stress.

8. CHEMICAL COMPOSITION OF STEEL REINFORCEMENT

Table 6 shows the chemical composition of the steel used as reinforcing for the test beams and also for some of the concrete ships and barges built by the Emergency Fleet Corporation. This table shows that the steel used in the test beams was generally much lower in carbon and much higher in phosphorus than that used in the ships and barges. It is generally believed that low carbon content reduces the resistance of the steel to fatigue, and that high phosphorus content reduces its resistance to shock. It has been stated that "in normal steels the strength against fatigue increases with the carbon content up to about 0.09 carbon," [10] and that "Phosphorus is especially dangerous in railroad rails which are exposed to severe shocks in service, * * *. For most purposes not more than 0.05 per cent of phosphorus is allowable in steel, except for thin rolled plates, etc." [11]

Beam 5L1 was the only one in which the chemical analysis of the steel approached that ordinarily expected in a good grade of reinforcing steel. Yet that beam failed under an unexpectedly small number of repetitions of reversed stress. This fact indicates that the poor grade of reinforcing used in the other beams is not in itself sufficient to account for the early failure of those beams.

9. EFFECT OF REST PERIODS ON NUMBER OF CYCLES THAT CAUSED FAILURE

Reference to Fig. 25 shows when rest periods occurred in the tests. There is no indication in Figs. 17 to 23 that these rest periods affected in any way the amount of the deformation. This result might be expected, but Fig. 26 seems to indicate that an increase in the proportion of the time during which the machine was not in operation caused a marked increase in the number of cycles which was required to cause failure. It is believed that the arrangement of the plotted points must be largely accidental,

[10] J. B. Johnson, Materials of Construction, fifth ed., p. 776.
[11] H. F. Moore, Materials of Engineering, p. 96.

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since there were inequalities in the stresses developed which would be expected to destroy the regularity of the curves.

It is not to be expected that a given period of rest will have equal value in prolonging the life of a beam, regardless of the stage of the test at which the rest period occurs.

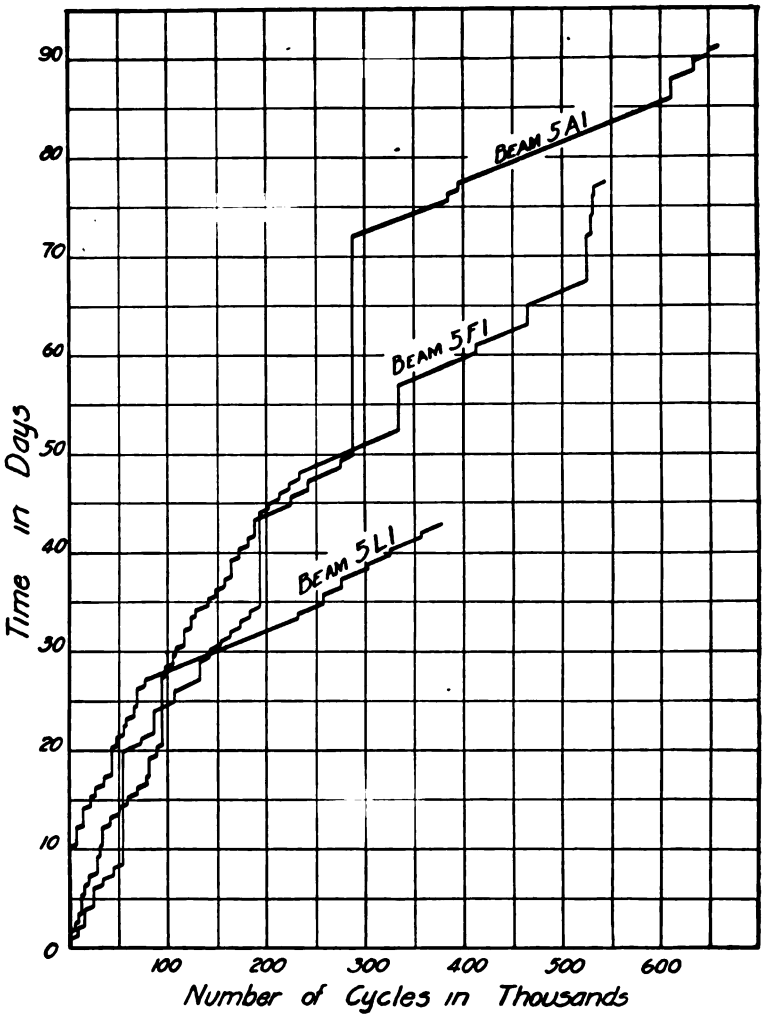

FIG. 25.—*Continuity of tests for beams 5A1, 5F1, and 5L1*

The rest periods also varied greatly in length and were irregular in their frequency.

When these facts are taken into consideration, it seems that less importance should be attached to the curves in Fig. 26 than their regularity would indicate.

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10. RELATION BETWEEN MAGNITUDE OF BENDING STRESSES AND NUMBER OF CYCLES THAT CAUSED FAILURE

In Fig. 27 the abscissas represent the number of cycles of load which caused failure in all beams except 5C1 and the ordinates

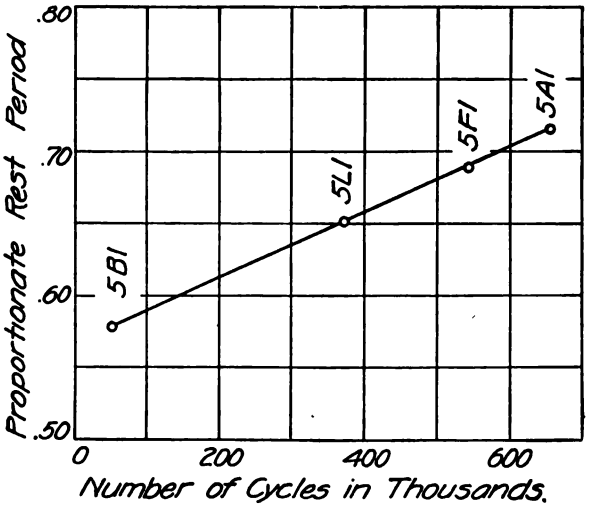

FIG. 26.—*Ratio of rest periods to total time of test*

represent the tensile stress in the reinforcing bars corresponding to the measured deformations at the same load. Fig. 28 is similar except that the ordinates represent the arithmetical sum

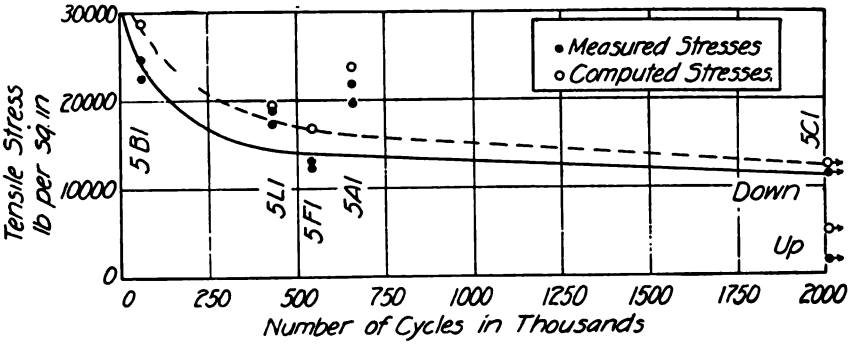

FIG. 27.—*Relation between tensile stress and number of cycles at failure*

of the tensile and compressive stresses instead of the tensile stresses alone.

The data of beam 5C1 are included in these figures although the test of that beam was not carried to the point of failure. They are used to indicate the trend of the curve although it is

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recognized that the inclusion of these data is not fully justified. In Fig. 27 the stresses for beam 5C1 for the upward load are given no weight in determining the positions of the curves. The upward load was much less than the downward load, and the stresses were correspondingly smaller. Therefore it is not likely that the time of failure would depend to any extent upon the number of repetitions of the upward load.

Although the discussion in Part IV, article 7, indicates that the tension failure of the steel in these beams can not have been due directly to the large number of repetitions of the tensile stress, nevertheless Figs. 27 and 28 indicate that there is some relation between the number of cycles of load required to cause failure and the tensile stress developed. The number of cycles

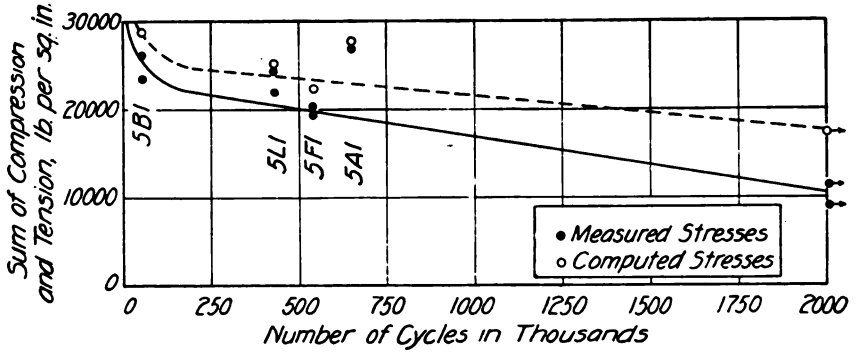

FIG. 28.—*Relation between sum of compressive and tensile stresses and number of cycles at failure*

decreased with the increase of stress, except in the case of beam 5A1.

In Part IV, article 3, it was suggested that the formation of the pronounced crack in the top flange of beam 5F1, and the consequent bending of the bar at this crack, may have been an important element in bringing about tension failure of the bar. If this explanation applied to all similar cases, bending would be likely to be more severe in beams where the tensile stresses are high than in those where they are low. This would help to account for the results shown in Figs. 27 and 28.

On the other hand, it seems that the arrangement shown in Figs. 27 and 28 may be largely accidental. In beam 5L1 large bars, 1¼ inches in diameter, were used and a slip of 0.06 inch at one end of one of the bars took place before failure occurred. Both of these facts would cause one to expect more severe stresses

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at the crack due to the bending of the bars than would be expected with small bars and with no end slip of the bars. It may be this action, rather than the severity of the tensile stress plotted in Fig. 27, which accounts for the early failure of this beam.

Beam 5B1 failed under the smallest number of applications of load, and both the calculated and measured stresses were higher in this beam than in any other. However, it has been pointed out in Part IV, article 5, that the early failure of this beam may have been due as much to the reduction in area caused by the gage holes in the small bars used as to the intensity of the tensile stress.

With the possibility of explaining the failures of beams 5B1 and 5L1 on other grounds than the tensile stress produced in the steel reinforcement, and with beam 5A1 not conforming to the suggested relation between intensity of stress and number of cycles which caused failure, it seems possible that the arrangement of plotted points shown in Figs. 27 and 28 may be accidental.

11. POSITION OF NEUTRAL AXIS

The positions of the neutral axis for the beams tested are shown in Figs. 29 to 33.

Fig. 19 gives the deformations in the reinforcing bars on the south side of beam 5B1 at the top and at the bottom of the beam. A study of this figure shows that up to about 50 000 cycles of load the amount of tension was nearly the same on the top with the load acting upward as that on the bottom with the load acting downward, and that the amount of compression was nearly the same on the top with the load acting downward as that on the bottom with the load acting upward. A similar statement may be made regarding the bars on the north side, Fig. 19. These relations are as should be expected when the upward loads and the downward loads are equal and when the reinforcing bars in the top and in the bottom of the beam are at equal distances from the neutral axis. The equality of stress in top and bottom bars under conditions of tension and compression, respectively, was not preserved, in general, after the number of load cycles had exceeded 50 000.

It would also be expected that the deformations at the top and at the bottom of the beam would be the same at any instant on the north side as those on the south side, but a study of Fig. 19 shows that they were not equal for beam 5B1. The differences, however, are such as to make the range in deformation from

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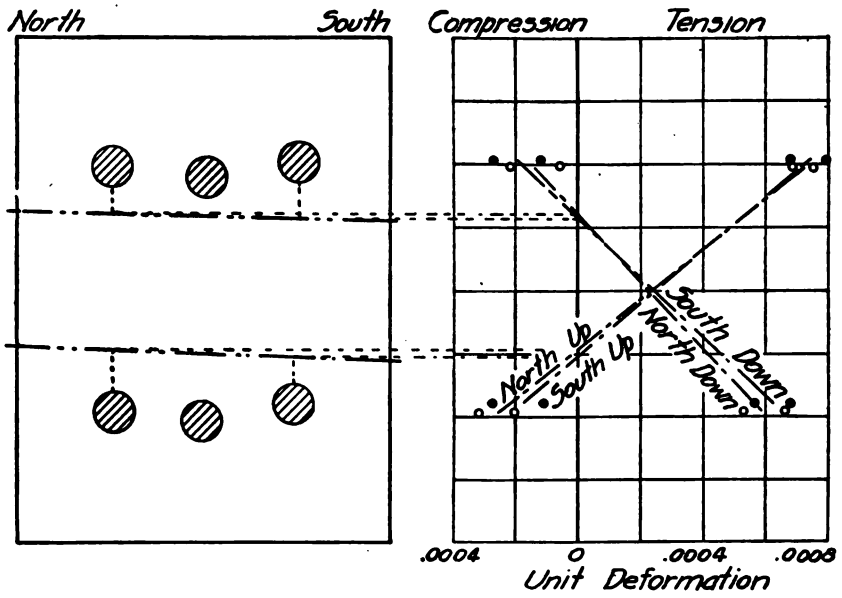

FIG. 29.—*Strain distribution and positions of neutral axis for beam 5A1*

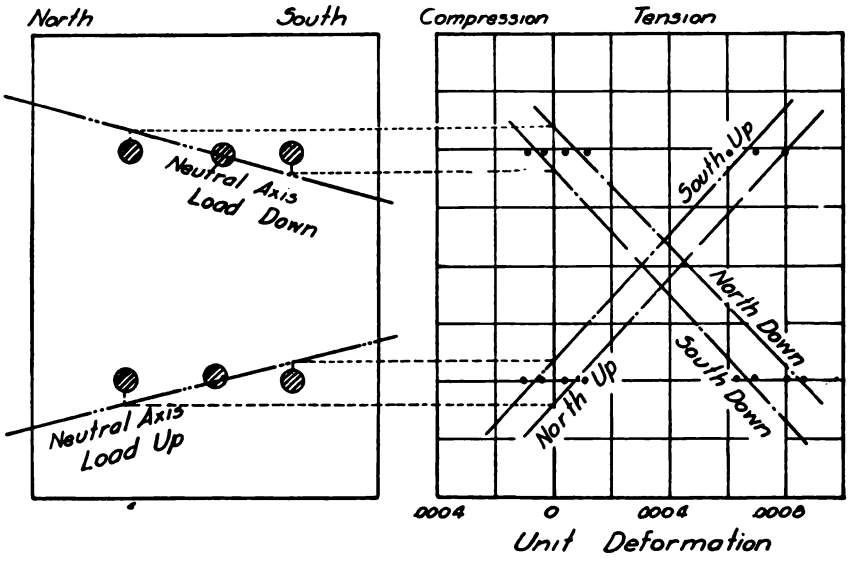

FIG. 30.—*Strain distribution and positions of neutral axis for beam 5B1*

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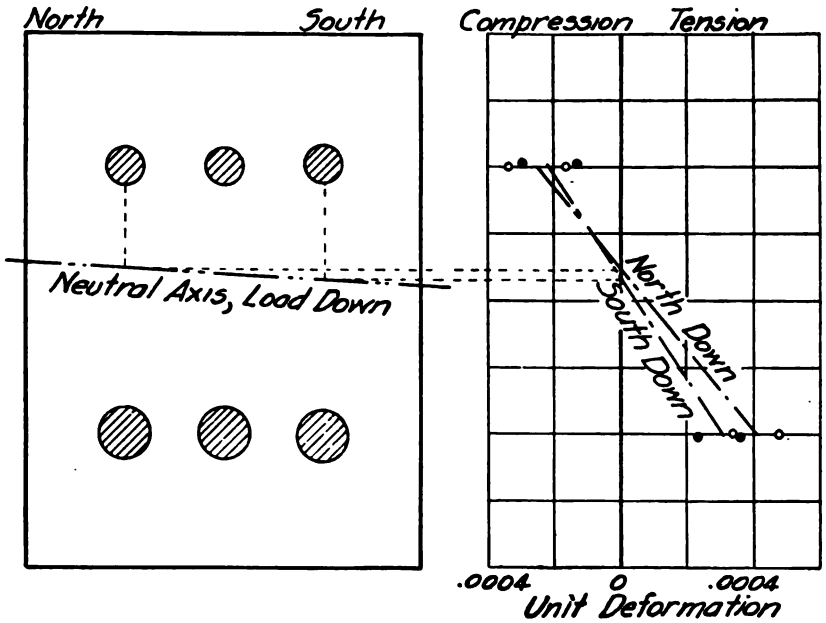

FIG. 31.—*Strain distribution and position of neutral axis for beam 5C1*

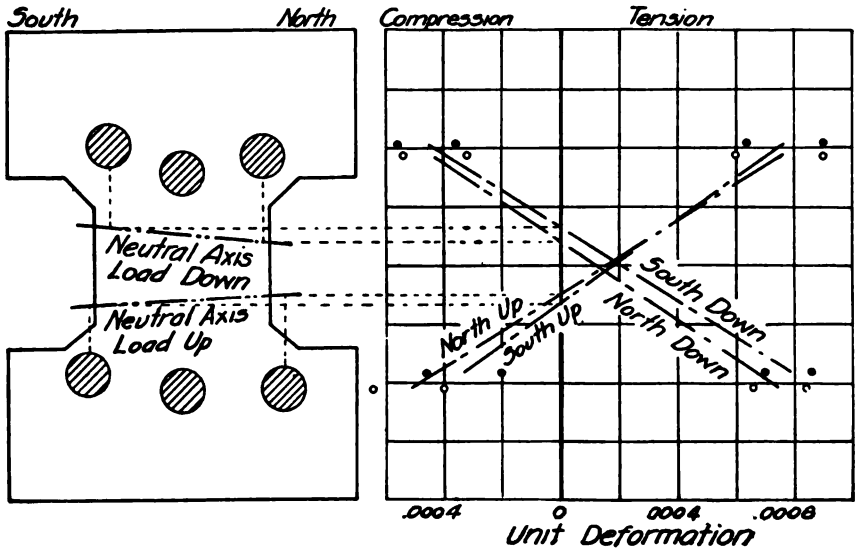

FIG. 32.—*Strain distribution and positions of neutral axis for beam 5F1*

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tension to compression at any one gage line, either on the top or on the bottom, the same as the total difference in deformations between a gage line on a top bar and one on a bottom bar. These relations are made much more clear by a study of the strain distributions for beam 5B1 that are shown in Fig. 30. The individual points plotted show the extreme range of deformations corresponding to load cycles above about 10 000 and below about 50 000. The lines are drawn through the mean values represented by the points.

From these strain distributions the approximate positions of the neutral axis for the upward loads and for the downward

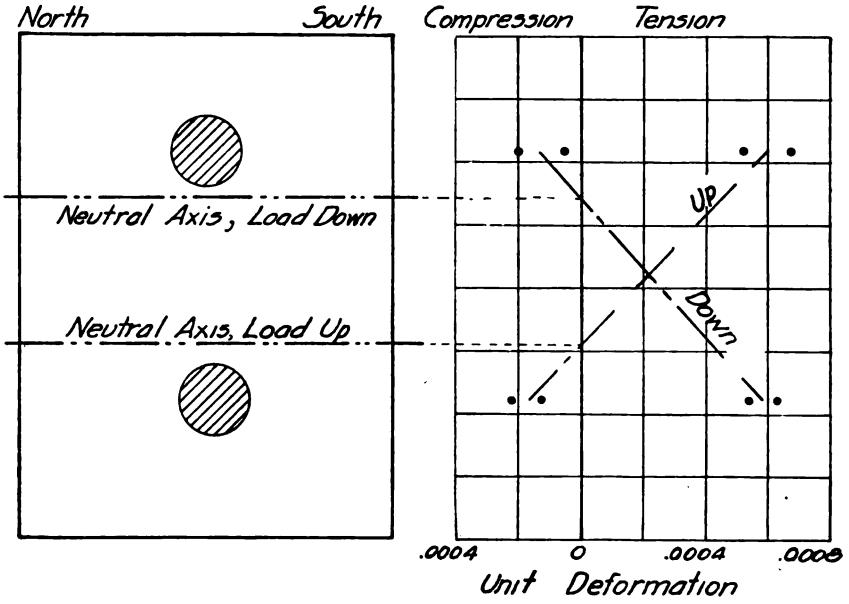

FIG. 33.—*Strain distribution and positions of neutral axis for beam 5L1*

loads have been determined and are shown in the same figure. The neutral axes under the conditions of upward and downward loading do not coincide, nor are they parallel, but the symmetry of the two neutral axes about the horizontal center line of the beam may be of some interest. The convergence of these axes could be accounted for by a curvature in the horizontal projection of the beam, but there is no evidence that any appreciable amount of warping sideways existed. However, a certain looseness in the bearings of the testing machine was observed, and it is possible that this introduced a slight inclination of the beam in one direction when the load was acting downward and in the opposite

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direction when the load was acting upward. This would bring about the conditions which were observed. Another possible explanation of the convergence of the neutral axes may be found in the lack of homogeneity in the concrete, which might have considerable effect on the elasticity of opposite sides of a beam of small cross section.

Similar diagrams are shown for beams 5A1, 5C1, 5F1, and 5L1. It should be noted that the strain readings in all of these cases were taken with the machine in operation, and it is not possible to show the instantaneous position of the neutral axis because readings could not be taken on all of the gage lines simultaneously. However, the deformation curves of Figs. 17 to 23 maintain nearly enough the same positions in relation to each other to indicate that by plotting the extreme range of deformations for a small number of cycles the average would represent with reasonable accuracy the conditions which existed at a given time. This procedure has been followed in preparing the diagrams shown in Figs. 29 to 33. The deformation measurements were taken on the side of a bar at the level of its center line.

The positions shown for the reinforcing bars in Figs. 29 to 33 were determined by measurements on the sections after the beams had been tested and sawed in two, except for beam 5C1. This beam has not been sawed in two, and the positions shown are those indicated in the design of the beam.

Any error of observation is likely to show itself in an apparent inclination of the neutral axis. In Figs. 29 and 31, representing beams 5A1 and 5C1, all of the inclination present may be due to errors of observation. Figs. 30 and 32, for beams 5B1 and 5F1, show marked inclination of the neutral axis, and it is likely that these diagrams represent approximately actual conditions in the beams.

For beam 5L1 the neutral axis, as shown in Fig. 33, is horizontal. In this beam there was only one bar in the top and one in the bottom and the measurements were taken on only one side of the beam; consequently, the data furnish no basis for determining the inclination of the neutral axis, if such existed.

The data for beam 5C1 for upward load seem inconsistent and have not been plotted in Fig. 31. The peculiar showing in Fig. 20, in which compression is sometimes found where tension should be expected, is not understood. The smallness of the upward load in relation to the downward load, the consequent importance

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of any residual deformations, and the importance, relatively, of small errors of observation were factors which probably contributed to the inconsistencies in the data.

12. SUMMARY

(*a*) For all of the beams tested failure was by tension in the steel. Generally the beams in which the highest stresses were developed in the test withstood a smaller number of repetitions of load than those in which the measured stresses were smaller. However, even the largest number of repetitions was so small that failure in the steel would not have been expected at the time at which it did occur considering that the observed stresses were so low. Other factors than the intensity of the tensile and compressive stresses seem to have had a part in bringing about the early tension failure.

(*b*) All of the tension failures in the reinforcing bars occurred at sections where large cracks extended entirely across the section of the beam. It is possible that in some cases the bending at these cracks was sufficient to make the bending of the bar an important factor in causing failure. The slipping of bars at the ends, such as happened in beam 5L1, would permit the opening of wide cracks and accentuate this tendency.

(*c*) The presence of the gage holes in the bars seems to have had some influence in hastening tension failure, but this influence is not very distinct.

(*d*) The quality of steel used for most of the reinforcement was poor, and this would contribute to bringing about early failure. However, this alone would not account for the small number of repetitions of stress generally required to produce failure.

(*e*) The effect of the repeated loading on the physical properties of the steel reinforcement was slight, at most; the indications of the presence of any effect probably is accidental.

(*f*) Although there appears to exist a marked effect of the rest periods on the life of the beam, it seems unreasonable to believe that the apparent effects are anything more than accidental.

(*g*) After 7000 repetitions of a bond stress of 161 pounds per square inch the slip at the end of the bar in beam 5L1 was less than 0.001 inch; that is to say, less than the amount which has been taken as the criterion of safe conditions based on tests of the bond resistance between concrete and steel. Yet, after 400 000 cycles of load, the amount of slip had increased so much that

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failure by slipping of the bars seemed imminent. It seems that the intervention of tension failure at an unexpectedly small number of repetitions of load prevented the bond failure of this specimen.

(*h*) Generally the neutral axes of the beams were inclined, suggesting a slight eccentricity in the application of the load. It is not apparent that the conditions causing this inclination of the neutral axis should have anything to do with shortening the life of beams under reversal of stress.

The tests reported herein indicate the importance of conducting further research upon the effect of repeated reversals of stress in reinforced beams.

WASHINGTON, July 9, 1920.

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BY

EDGAR BUCKINGHAM, Physicist

Bureau of Standards

DECEMBER 20, 1920

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NOTES ON SMALL FLOW METERS FOR AIR, ESPECIALLY ORIFICE METERS[1]

By Edgar Buckingham

ABSTRACT

The paper contains information, compiled for the use of physiologists of the Chemical Warfare Service, on the selection, design, and properties of small flow meters for air.

1. FORM OF ORIFICE

If an orifice meter is adopted, it is not advisable to use a very thin orifice plate, nor, if the plate is thick, to chamfer it on the outside so as to give the orifice a sharp edge. The thickness of the plate should be about one and one-half times the diameter of the orifice, and after a hole of the required diameter has been drilled through the plate it should be rounded off to a trumpet shape on the inlet side, so as to give an easy entrance. A suitable profile is a quarter circle of radius slightly less than the thickness of the plate, set tangent to the hole already drilled and to the entrance face of the plate, thus leaving the orifice cylindrical for a short distance in from the exit face.

An orifice of this sort, if carefully made and smoothly finished, will give a discharge which is within a few per cent of the so-called "theoretical" discharge. This is convenient because the size of orifice needed for a particular purpose may then be computed approximately from the theoretical equations, which is not the case for sharp-edged orifices.

[1] These notes were prepared in December, 1917, at the request of the Chemical Warfare Service and were intended primarily for the use of physiologists. Various calls for the same sort of information have made it seem desirable to publish them.

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2. COMPUTATION OF THE THEORETICAL DISCHARGE THROUGH A FRICTIONLESS ORIFICE

Let—

D = the diameter of the orifice in millimeters.

h = the pressure drop at the orifice.

p = the pressure of the air on the inlet side, or the "initial pressure."

$p_1 = p - h$ = the pressure on the exit side, or the "back pressure."

$r = p_1 \div p$. It is assumed that the pressure drop h is not more than 0.4 p, so that the pressure ratio r is always between 0.6 and 1.0.

T = the absolute temperature of the air on the inlet side, or the "initial temperature," measured in centigrade degrees.

V = the rate of discharge of dry air in liters per minute, measured under the initial conditions p, T.

Then the theoretical discharge is given by the equation: [2]

$$V = 2.12\, D^2 \sqrt{T} \sqrt{r^{\frac{10}{7}} - r^{\frac{12}{7}}} \qquad (1)$$

It may be noted that the absolute values of the pressures do not appear explicitly in this equation. If the initial and final pressures are both doubled, the *mass* of air discharged per minute will also be doubled; but since V is now to be measured at this doubled initial pressure, its value in liters per minute will remain unchanged and may still be found from equation (1).

The volume which these V liters of air would occupy at any other temperature and pressure may be found by means of the familiar relation $pV = RT$. In particular, the volume V_0 which the air discharged per minute would occupy if measured in liters at 0° C and at a pressure of 760 mm of mercury, is given by the equation

$$V_0 = 0.761\, D^2 \frac{p}{\sqrt{T}} \sqrt{r^{\frac{10}{7}} - r^{\frac{12}{7}}} \qquad (2)$$

in which p is expressed in millimeters of mercury.

[2] A deduction of this equation is given in a note at the end of the paper.

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3. THE RELATION OF ACTUAL TO THEORETICAL DISCHARGE

In practice, the discharge of dry air from an orifice or nozzle of the form described in section 1 is found to be a little less than the value computed from equation (1) or (2). The ratio of the actual to the theoretical discharge is known as the discharge coefficient. For orifices of a diameter of 2 mm or more the *dsicharge coefficient* will usually be over 0.9 and may be nearly 1. Hence the theoretical equation enables us to computé beforehand about what size of orifice will be needed for a given discharge under given conditions, and so to design or to select an orifice suitable for the purpose in hand.

But it is not possible to predict the value of the discharge coefficient exactly; and, furthermore, the value is not constant for a given orifice, but varies with the pressure ratio r. Hence each orifice must be standardized by experiments at various values of r. For orifices which are large enough that accurate reproduction is mechanically practicable, a single standardization may suffice for a number of similar orifices; but for diameters of only 1 or 2 mm such accuracy is difficult of attainment, and should not be assumed without investigation.

It is not possible to give exact formulas for moist air; but it may be said that, unless the air is so moist that water is deposited in the orifice, the discharge will not differ much from that for dry air. If water is deposited on the walls of the orifice, the discharge rate is liable to be irregular. For security, the air should be dry, when practicable, or the orifice should be slightly warmed to prevent the deposition of water upon it.

4. THE EFFECT OF VARIATIONS IN THE INITIAL TEMPERATURE OF THE AIR

While the discharge coefficient of a given orifice varies somewhat with the pressure ratio, it is not sensibly affected by moderate changes in the initial temperature of the air; and for an initial temperature range of 0 to 50° C it is safe to treat the discharge coefficient as independent of the temperature. This means that when an orifice has been standardized at one temperature, the same standardization may be used for other temperatures if the changes of temperature are allowed for as indicated in the theoretical equations already given. If the air reaching the orifice is at ordinary room temperature, in the vicinity of 20° C or 293°

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absolute, a change of 1° C changes the value of V or of V_0 by a trifle more than one-eighth of 1 per cent. Hence, if no greater accuracy than 1 per cent is required, an orifice that has been standardized at 1 temperature—e. g., 18° C—may be used without any temperature correction for any temperature within 5 or 6° of the temperature at which the experimental standardization was carried out.

5. THEORETICAL DISCHARGE CURVE IN TERMS OF THE SUCTION HEAD

In using an orifice meter the quantity primarily observed is the pressure drop h; the initial temperature and either the initial or the back pressure being also observed if not already known. Having the values of h and either p or p_1 we may set $r=\frac{p-h}{p}$ or $r=\frac{p_1}{p_1+h}$ and compute the theoretical discharge by means of equation (1).

But since the computation is rather cumbersome, it is convenient, if many problems relating to the same initial conditions are to be solved, to construct once for all a curve giving V in terms of h for those conditions. Furthermore, since D enters the equation very simply, it is well to construct the curve for $D=1$ mm and allow separately for changes in D.

Let us therefore set $D=1$; and let us adopt, as our fixed initial conditions, 18° C and 1 atmosphere pressure, the air being drawn through the orifice by the suction h and the back pressure being less than atmospheric. We now have $T=291$, and equation (1) reduces to

$$V=36.1\sqrt{r^{1.8}-r^{1.4}} \tag{3}$$

Let us suppose, further, that the pressure drop h is measured in centimeters of water. Then, since a head of 760 mm of mercury is equivalent to a head of 1034 cm of water, we have to substitute in equation (3) the value $r=(1034-h)/1034$. By using various values of h we may then compute the corresponding values of the theoretical discharge V and plot a curve showing the relation of V to h for an orifice of 1 mm diameter under the given initial conditions. Such a curve is appended to this paper.

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6. ILLUSTRATIONS OF THE USE OF THE THEORETICAL DISCHARGE CURVE

The method of using such a curve may be illustrated by the following examples:

(*a*) Suppose we wish to know the rate at which air under the above-described initial conditions will be sucked through a 2.6 mm orifice by a head of 27 cm of water. Reading from the curve at $h=27$ we find $V=3.08$. An orifice of 2.6 mm diameter will therefore give a theoretical discharge of $3.08\times 2.6^2=20.8$ liters per minute. Actually, the discharge will be a few per cent less than this.

(*b*) Suppose that in the foregoing example the initial temperature had been not 18 but 30° C; then the theoretical rate of discharge in liters per minute *measured at 30° C* would have been

$$20.8\sqrt{\frac{273+30}{273+18}}=21.2.$$

(*c*) Suppose that we want to select an orifice which shall discharge 85 liters per minute with a suction head of from 40 to 70 cm of water. From the curve we find that for $D=1$, at $h=40$ $V=3.72$, and at $h=70$ $V=4.84$. We must therefore use an orifice of such diameter D that $\frac{85}{D^2}$ lies between 3.72 and 4.84, which means that D must lie between 4.78 and 4.19 mm. An orifice of 4.5 mm diameter would give 85 liters per minute at the same head as would be required for an orifice at 1 mm diameter to discharge $\frac{85}{4.5^2}=4.2$ liters per minute; and we find from the curve that the required theoretical head is $h=52.2$ cm of water.

7. REMARKS

It will be sufficiently evident from the foregoing examples how the curve may be used for other temperatures than the one for which it was constructed, as well as for other diameters than 1 mm. But since the use at present proposed for the curve is merely for selecting or designing orifices which are then to be standardized by experiment, and since the discharge coefficient will always be in doubt by several per cent until this standardization has been effected, no great accuracy in using the curve is needed, and allowances for temperature changes may as well be disregarded so long as the initial temperature remains between 0 and 35° C.

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To the degree of approximation now in question the curve may also be used for solving problems relating to the discharge of air against a constant back pressure of one atmosphere, the head h cm of water being now an excess pressure on the inlet side above one atmosphere. At $h=80$, the value of V read from the curve is about 3.5 per cent greater than the theoretical discharge under these conditions if the volume is measured at 18° C and at the initial pressure of (1 atm. $+h$); and it is about 4.5 per cent less than if the volume is measured at 18° C and at 1 atmosphere. For smaller values of h these discrepancies are smaller; hence for rough purposes the curve may be used to solve problems relating to this second method of working.

If, finally, readings are desired in terms of V_0, the volume discharged measured under standard conditions, a curve may easily be constructed from equation (2). The time-consuming part of such work is the computation of $\sqrt{r^{\frac{10}{7}}-r^{\frac{12}{7}}}=B$. A table of values of B in terms of r from $r=0.92$ to $r=1.00$ is given at the end of the paper.

8. EFFECT OF THE SPEED OF APPROACH

The theoretical equations given above are deduced on the assumption that the initial velocity of the air, at the place where p and T are measured, is so small that its square is negligible in comparison with the square of the speed of the air through the orifice. Hence if the orifice plate is merely inserted as a diaphragm across a pipe which is not much larger in diameter than the orifice itself, the formulas as given will be considerably in error.

If the pressure drop at the orifice is small, so that the pressure ratio is not far from unity, the density of the air is not very different on the two sides of the orifice. Hence the speeds in the approach pipe and in the orifice will be approximately inversely proportional to the areas, and their squares, to the fourth powers of the diameters, so that it is not necessary to slow down the approaching air by introducing a large chamber at the place where the orifice is to be put. If, for example, the internal diameter of the housing is 5 times that of the orifice, the square of the speed through the orifice is some 625 times the square of the speed of approach, and a further increase of the diameter ratio could not have any appreciable effect on the rate of flow through the orifice.

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9. REMARKS ON SOME OTHER SIMPLE FORMS OF FLOW METER

(*a*) SHARP-EDGED ORIFICES or holes in thin plates are often used in flow meters, instead of the trumpet-shaped orifices or nozzles recommended in section 1 above, and if properly standardized they are satisfactory. They have, however, the disadvantage that the discharge coefficient varies rapidly with the pressure ratio and may be as small as 0.6, so that the theoretical equations can not safely be used to give an approximate estimate of the rate of discharge by merely assuming a constant discharge coefficient of, say, 0.95, as may be done with an orifice which has a trumpet-shaped entrance.

(*b*) THE VENTURI METER, while excellent in large sizes, could hardly be made satisfactory for rates of flow of the order of 80 liters per minute or less; and it would be difficult to construct on so small a scale as would be required unless recourse were had to the use of extremely delicate differential gages with their attendant disadvantages.

(*c*) CAPILLARY TUBE METERS in which the pressure drop to be measured is due to the resistance of a length of tube, have some defects, of which the most serious is liability to obstruction by dust or condensed water vapor. For small rates of flow the tube must be either very long or very fine. Long tubes are inconveniently fragile if made of glass, while, if the tube is opaque, the presence of an obstruction can be detected only from the behavior of the tube. On the other hand, the fineness of bore needed when the tube is short, greatly increases the liability to obstruction. And though the visibility of an obstruction in a glass tube may save the observer from relying on erroneous readings of flow, it does not remove the obstruction nor obviate the necessity of either cleaning or restandardizing.

Certain other points regarding capillary tube meters may also be worth mentioning. We shall first suppose that the tube is straight, and that its length is a large multiple of its diameter (e. g., 1000) so that the resistance is only slightly influenced by the nature of the ends and is nearly proportional to the length of the tube. Such a tube may behave in either of two quite different ways.

If the flow is slow enough, the rate of discharge is directly proportional to the pressure drop—a very simple and convenient relation; but it is also inversely proportional to the viscosity of

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the air in the tube; and since in the vicinity of room temperature the viscosity of air increases by about 0.3 per cent per degree C rise in temperature, no great accuracy can be achieved unless the tube is jacketed in some way, so that its temperature can be controlled and observed.

On the other hand, if the flow is very rapid, the nature of the fluid motion is entirely different; the resistance is greater and the rate of discharge increases more nearly as the 0.6 or 0.5 power of the pressure drop. And, finally, there is an ill-defined intermediate range of speeds where either of the foregoing regimes may establish itself, the nature of the motion and the relation of discharge to pressure drop sometimes changing suddenly from the first to the second. In this critical region no standardization can be relied on, and care must be taken, not only to avoid this range, but also to make sure that a standardization under one regime is not, by inadvertence, extrapolated to a point where the other regime is the actual one.

Short tubes form a transition stage between tubes which are so long that the nature of the ends is unimportant, and orifices in thin plates where only the ends remain and the middle has shrunk to nothing. We have very little information about the behavior of such tubes or even of long tubes which are not straight and cylindrical. It does not follow that such tubes may not be entirely satisfactory for use as flow meters after they have been standardized by experiment. But from the standpoint of design, they present the disadvantage that we have no simple mathematical theory which we know will represent their behavior sufficiently well to enable us to select the dimensions needed for a particular purpose with a certainty of getting approximately the desired result.

10. THE MEASUREMENT OF VERY SMALL RATES OF FLOW

For an orifice of 1 mm diameter and for the initial conditions, 18° C and 1 atmosphere pressure, we find by reading from the curve that a discharge of 1 liter per minute requires, theoretically, a suction head of about 2.8 cm of water. And since this is too small a head to be read accurately on an ordinary vertical U gage, it is evident that for rates of 1 liter per minute, or less, a more sensitive gage must be used, or, if an orifice meter is to be employed, the orifice must be less than 1 mm in diameter.

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The sensitiveness of the reading of h may be increased about 10 times by means of an inclined U gage; but if much more is required, recourse must be taken to some more sensitive type of differential gage. There is no difficulty in making such gages, but they are likely to be inconvenient and sluggish; and unless both sides of the gage are equally accessible to slight accidental irregularities of pressure, such as may arise from a gusty wind or the opening and closing of doors, very sensitive gages are not easy to work with. On the other hand, orifices of much less than 1 mm diameter, even when carefully finished, are so liable to be affected by dust, dirt, or condensed water vapor that they are not to be recommended.

In view of the fact that the resistance of a tube of given diameter can be indefinitely increased by increasing the length, it seems likely that the most convenient form of meter for low rates of flow will be one in which the resistance across which the pressure drop is observed is furnished by a capillary glass tube, a long tube in coil form being preferred to a short and very fine straight tube. If the air is carefully dried and freed from dust, such a tube should be satisfactory in the sense of providing a constant resistance.

If it is not practicable or not desirable to dry the air before it reaches the meter, condensation in the tube may be prevented by keeping the tube at a temperature considerably above that of the incoming air. As already remarked in section 9(*c*), the temperature of the tube must be controlled if accurate measurements are to be made, and, if the bath in which the tube is immersed is kept well above the temperature of the incoming air, there will be no risk of condensation in the tube.

If a meter of this sort is adopted, the dimensions of the tube may be chosen so as to give a convenient pressure drop h at the desired rate of flow. Equations might be given connecting the rate of flow with the pressure drop for straight, round tubes of known dimensions, but they would not be reliable for bent or coiled tubes. In any event, such a tube must be standardized by experiment; and, if apparatus for standardization is available, it is a simple matter to select by trial a suitable tube for the purpose in hand.

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It is advisable to keep the discharge rate always well below the critical range mentioned in section 9(*c*); that is, to have the flow always so slow that the discharge rate is nearly proportional to the pressure drop *h*.

WASHINGTON, December 6, 1920.

Values of $B=\sqrt{r^{1.9}-r^{1.9}}$

h	$\frac{1034-h}{1034}=r$	$\sqrt{r^{1.9}-r^{1.9}}=B$	*h*	$\frac{1034-h}{1034}=r$	$\sqrt{r^{1.9}-r^{1.9}}=B$
80	0.9226	0.1424	10	0.9903	0.0523
60	.9421	.1247	7	.9932	.0438
40	.9613	.1029	4	.9961	.0332
30	.9710	.0896	2	.9981	.0235
20	.9807	.0736	1	.9990	.0166
15	.9855	.0639	0.5	.9995	.0118

NOTE

DEDUCTION OF THE "THEORETICAL EQUATION"

Let a fluid be flowing steadily along a channel from a section A, where the static pressure is p and the absolute temperature is T, to a second section A_1, where the pressure and temperature are p_1 and T_1, the sections being at the same level so that gravity may be disregarded.

Let v, K, e be, respectively, the volume, the kinetic energy, and the internal energy, of unit mass of the fluid as it passes A; and let symbols with subscripts refer to A_1. Let Q be the heat received from without by unit mass while it is between A and A_1.

By the first law of thermodynamics, the increase of energy per gram of fluid from A to A_1 is equal to the heat added plus the excess of the work done on the fluid as it enters at A over the work it does on the fluid ahead of it in issuing at A_1. Hence we have

$$(K_1+e_1)-(K+e)=Q+pv-p_1v_1,$$

Or

$$K_1-K=e-e_1+pv-p_1v_1+Q \qquad (1)$$

Let A_1 be the section of a stream issuing from a nozzle or orifice, and let A be a section farther upstream and of so much larger area that the speed at A is small, and the kinetic energy K therefore negligible. Let us also suppose that Q is negligible and the flow sensibly adiabatic. Then equation (1) takes the simpler form

$$K_1=e-e_1+pv-p_1v_1 \qquad (2)$$

This is general and applicable to any fluid.

Now let the fluid be ideal gas—that is, one for which the equations

$$\left.\begin{aligned} pv&=RT\\ C_v&=const\\ e&=TC_v+const\\ C_p&=C_v+R \end{aligned}\right\}$$

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are satisfied. Within the limits of accuracy needed here, air is such a gas. By these equations we may easily reduce equation (2) to the form

$$K_1=C_p(T-T_1), \tag{4}$$

which says that the kinetic energy acquired, per gram, is equal to the product of the fall in temperature by the specific heat at constant pressure.

Let S_1 denote the arithmetical mean speed over the section A_1. If the speed were uniform all over A_1 and there were no cross currents or eddies, we should evidently have

$$K_1=\tfrac{1}{2}S_1^2 \tag{5}$$

and by (4)

$$S_1=\sqrt{2C_p(T-T_1)}. \tag{6}$$

The agreement of experiment with deductions from equation (6) is close enough to show that equation (5) is nearly fulfilled under ordinary conditions, and equation (5) is therefore adopted as an assumption, though often not mentioned as such.

Up to this point it has not been assumed that the flow was frictionless. If there is resistance due to viscosity or turbulence, the fluid will be heated by the dissipation; T_1 will be raised and S_1 diminished; but equation (6) will remain satisfied.

In short, well-formed nozzles or orifices the resistance is small, and if we neglect it altogether we may treat the expansion of the gas as not only adiabatic, but isentropic, and may then apply the familiar equations for isentropic expansion of an ideal gas, viz:

$$pv^k=\text{const};\quad T=p^{\frac{k-1}{k}}\times\text{const} \tag{7}$$

where $k=C_p/C_v$. By the second of these we have

$$\frac{T_1}{T}=\left(\frac{p_1}{p}\right)^{\frac{k-1}{k}}=r^{\frac{k-1}{k}} \tag{8}$$

So that (6) may be written in the form

$$S_1=\sqrt{2TC_p\left(1-r^{\frac{k-1}{k}}\right)} \tag{9}$$

an equation which gives the speed S_1 of the frictionless jet in terms of the initial temperature T, the specific heat at constant pressure C_p, the pressure ratio $p_1/p=r$, and the specific heat ratio k.

The volume of gas passing A_1 in unit time is $V_1=A_1S_1$; and the volume of this same mass measured under the initial conditions is

$$V=V_1\frac{v}{v_1}=A_1S_1\frac{v}{v_1}$$

or by (9)

$$V=A_1\frac{v}{v_1}\sqrt{2TC_p\left(1-r^{\frac{k-1}{k}}\right)} \tag{10}$$

Let A_1 be a circle of diameter D, and by (7) set $v/v_1=r^{\frac{1}{k}}$. Then substituting in (10) we have

$$V=\frac{\pi}{2\sqrt{2}}D^2\sqrt{TC_p\left(r^{\frac{2}{k}}-r^{\frac{k+1}{k}}\right)} \tag{11}$$

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This holds for any normal units. For example: If D is in cm, T in °C, and C_p in ergs per g. per °C, V will be in cm³ per second.

For dry air at ordinary pressures and temperatures we have $k=1.40$, $C_p=1.01\times 10^7$ erg/g/°C, so that (11) takes the form

$$V[\text{cm}^3/\text{sec.}]=3530\, BD^2[\text{cm}^2]\sqrt{T°C} \tag{12}$$

where

$$B=\sqrt{r_7^{10}-r_7^{12}} \tag{13}$$

For V in liters per minute and D in millimeters, this reduces to

$$V[\text{liters/min.}]=2.12\, BD^2[\text{mm}^2]\sqrt{T°C}, \tag{14}$$

which is given as equation (1) in the body of the paper.

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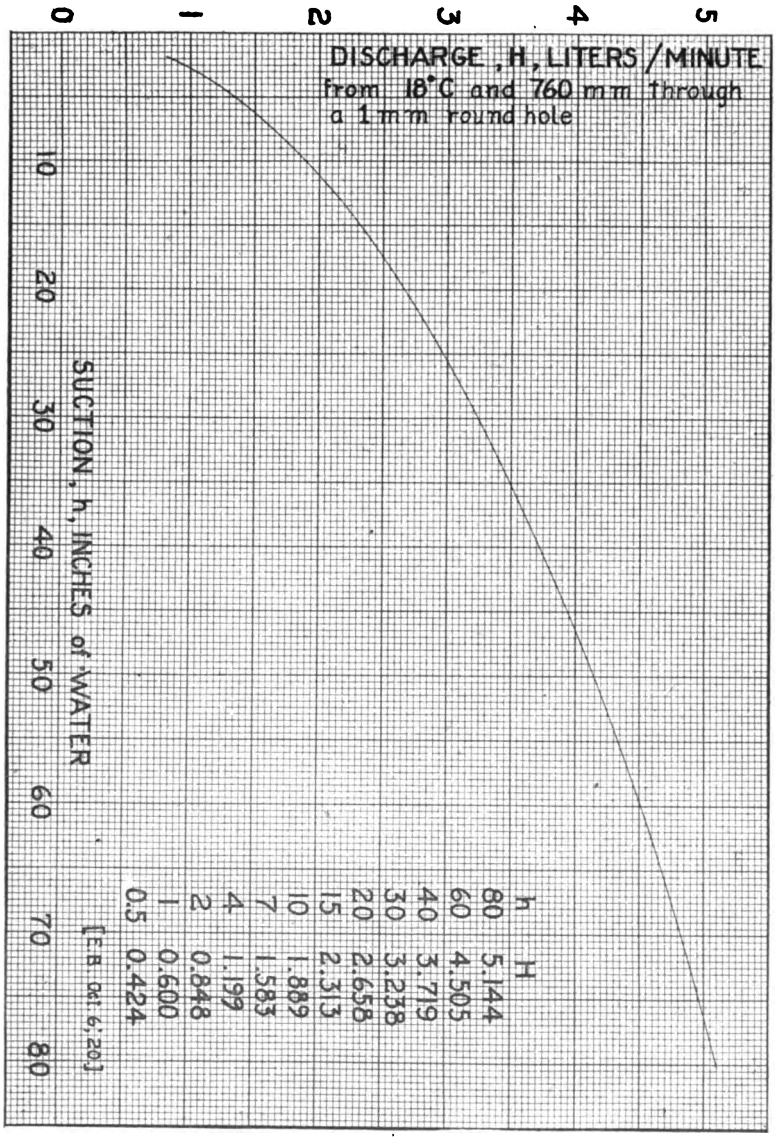

FIG. 1.—*Discharge curve*

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 184

FIRE TESTS OF BUILDING COLUMNS

BY

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APRIL 21, 1921

PRICE, 75 CENTS

Sold only by the Superintendent of Documents, Government Printing Office
Washington, D. C.

WASHINGTON
GOVERNMENT PRINTING OFFICE
1921

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PREFACE

The investigation herein reported was conducted in cooperation with the Associated Factory Mutual Fire Insurance Companies and the National Board of Fire Underwriters, the tests being made at Underwriters' Laboratories, Chicago, Ill.

When the assistance of this Bureau on certain portions of the work was sought during the early part of 1914, the preparatory work had been in progress for about four years, a building adapted for the tests had been erected, and the design of equipment and test columns nearly completed.

As the safety of building supports is a matter of general public interest, participation in the proposed investigation appeared justified, and a basis of cooperation was arranged whereby the work was made a part of the investigation on the fire-resisting properties of building materials that was then being initiated by this Bureau.

By the method of presentation that follows, Section I gives a general outline of the tests; Sections II, III, and IV give descriptions of the columns, column coverings, and methods used in their preparation; Section V and Appendix D, results of auxiliary tests of materials; Sections VI, VII, VIII, and IX, descriptions of apparatus and method of testing; Sections X and XI and Appendices A, B, C, results of tests; Sections XII and XIII, discussion of test data and conclusions.

The full presentation of the results of the present investigation is necessarily of considerable extent, and familiarity with the general plan will be of material aid in reading and, if desired, in selecting for particular attention such portions as may best serve the reader's immediate purpose.

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FIRE TESTS OF BUILDING COLUMNS

By S. H. Ingberg, H. K. Griffin, W. C. Robinson, and R. E. Wilson

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I. INTRODUCTION

1. PURPOSE

The purpose of this investigation is to ascertain (1) the ultimate resistance against fire, of protected and unprotected columns as used in the interior of buildings; (2) their resistance against impact and sudden cooling from hose streams when in a highly heated condition.

While columns form the most important element in the strength of a building, few representative tests have been made to determine their ability to support load when exposed to fire, and fire experience has only a limited value due to the many unknown variables involved. As a consequence, wide differences in requirements relating to the protection of columns against fire exist between different municipal codes and other published regulations. This investigation was undertaken to obtain information on which proper requirements for the more general types of columns and protective coverings can be based.

2. SCOPE OF TESTS

The present series consists of 106 tests of columns, of which 91 were fire tests and 15 fire and water tests.

The fire test series includes (1) tests of representative types of unprotected structural steel, cast iron, concrete-filled pipe, and timber columns; (2) tests wherein the metal was partly protected by filling the reentrant portions or interior of columns with concrete; (3) tests wherein the load-carrying elements of the columns were protected by 2-inch or 4-inch thicknesses of concrete, hollow clay tile, clay brick, gypsum block, and also single or double layer of metal-lath and plaster; (4) reinforced-concrete columns with 2-inch integral concrete protection.

The covering materials for each class of protection were obtained from the main producing regions of the country, the object being to include samples from the principal mineralogical subdivisions that find general application in building construction. A large number of auxiliary tests of constituent materials were made, including several hundred compression tests of the concrete employed.

The test columns were designed for a working load of approximately 100 000 pounds, as calculated according to accepted formulas, the amount varying somewhat for the different sections.

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The load was maintained constant on the column during the test, the efficiency of the column or its covering being determined by the length of time it withstood the combined load and fire exposure.

The latter was produced by placing the column in the chamber of a gas-fired furnace whose temperature rise was regulated to conform with a predetermined time-temperature relation. Measurements were taken of the temperature of the furnace and test column and of the deformation of the latter due to the load and heat.

In the fire and water tests the column was loaded and exposed to fire for a predetermined period, at the end of which the furnace doors were opened and a hose stream applied to the heated column, the duration of the application and the pressure at the nozzle varying with the length of time the corresponding type of column withstood the regular fire tests.

3. ACKNOWLEDGMENTS

The tests were jointly conducted by the Associated Factory Mutual Fire Insurance Companies, the National Board of Fire Underwriters, and the Bureau of Standards. The fire tests were made at Underwriters' Laboratories, Chicago, Ill. The furnace and related equipment were designed and constructed by Underwriters' Laboratories during the period 1912 to 1917, the work being coordinated with a general building program that was carried out by them at that time. Its use, except for repairs and replacements, was donated for the tests. Preliminary work relative to the testing schedule was begun as early as 1910 by the Associated Factory Mutual Fire Insurance Companies and Underwriters' Laboratories, the cooperation of this Bureau beginning in 1914. A final schedule of tests was adopted in March, 1916. The preparatory work of covering and placing the test columns extended from May, 1916, to May, 1917. With the completion of the equipment, testing of columns began in June, 1917, and was completed in December, 1918. This paper in its final form was approved by the cooperating parties in December, 1920.

The apparatus for measuring temperature and deformation was supplied by this Bureau. Auxiliary physical and chemical tests of the materials of the test columns and their protective coverings were made at the Washington and Pittsburgh laboratories of the Bureau.

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The expense for labor and material was shared on an equal basis by the three cooperating units. The materials were in part purchased under the usual specifications, and in part supplied gratis by the following companies, whose cordial cooperation is herewith acknowledged:

American Bridge Co., Chicago, Ill., steel test columns.
Bethlehem Steel Co., South Bethlehem, Pa., steel test columns.
Chicago Bridge & Iron Works, Chicago, Ill., steel test columns.
R. D. Cole Manufacturing Co., Newnan, Ga., steel test columns.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa., steel test columns.
U. S. Cast Iron Pipe & Foundry Co., Burlington, N. J., cast-iron test columns.
Lally Column Co., Cambridge, Mass., steel-pipe test columns.
National Lumber Mfrs. Assn., Chicago, Ill., timber test columns.
Southern Pine Assn., New Orleans, La., timber test columns.
West Coast Lumbermen's Assn., Seattle, Wash., timber test columns.
Corrugated Bar Co., Buffalo, N. Y., reinforcing steel.
Associated Metal Lath Mfrs. Chicago, Ill., expanded metal lath.
Zander-Reum Co., Chicago, Ill., woven-wire lath.
Chicago Portland Cement Co., Chicago, Ill., Portland cement.
Marblehead Lime Co., Chicago, Ill., lime.
Pelee Island Sand & Gravel Co., Cleveland, Ohio, sand.
Phoenix Sand & Gravel Co., New York, N. Y., sand.
American Sand & Gravel Co., Chicago, Ill., sand and gravel.
Chicago Gravel Co., Chicago, Ill., sand and gravel.
Union Sand & Material Co., St. Louis, Mo., sand and gravel.
Haverstraw Crushed Stone Co., New York, N. Y., crushed stone.
Ohio Quarries Co., Cleveland, Ohio, crushed stone.
Rockport Granite Co., Rockport, Mass., crushed stone.
White Fireproof Construction Co., New York, N. Y., hard coal cinders.
Camp Conduit Co., Cleveland, Ohio, hollow clay tile.
National Fireproofing Co., Pittsburgh, Pa., hollow clay tile.
Whitacre Fireproofing Co., Waynesburg, Ohio, hollow clay tile.
Gypsum Fireproofing Co., Chicago, Ill., gypsum block.
Keystone Fireproofing Co., New York, N. Y., gypsum block.
Bestwall Manufacturing Co., Chicago, Ill., gypsum wall board.

The participation of the Associated Factory Mutual Fire Insurance Companies was under the direction of H. O. Lacount, assistant secretary and engineer, and they were represented on the preliminary conference and preparatory work by C. W. Mowry, and at different times in conducting the tests and preparing the results for publication, by R. E. Manning, W. G. Lawrence, and R. E. Wilson.

The National Board of Fire Underwriters participated through Underwriters' Laboratories, the work being under the direction of W. C. Robinson, vice president, assisted by F. W. Frederick, W. G. Howell, and F. Taylor in designing and constructing the testing furnace and related equipment, and at different times by G. W. Riddle and R. K. Porter in conducting the tests.

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The cooperation of this Bureau was under the administrative direction of C. W. Waidner, chief of the division of heat and thermometry. S. H. Ingberg and H. K. Griffin were actively associated with the work for the periods 1914 to 1920 and 1917 to 1920, respectively, the former being in direct charge of the experimental program.

Technical assistance for extended periods was given by A. J. Steiner and R. F. Zeunnert.

Mineralogical analyses of concrete aggregates were made by R. S. Knappen of the department of geology of the University of Chicago.

Acknowledgment is also due to a number of engineers, contractors, architects, and public officers who kindly examined a preliminary draft of the schedule of tests which was submitted to them. Their criticisms and suggestions were duly considered in formulating the final program.

II. DESCRIPTION OF COLUMNS

1. STRUCTURAL STEEL COLUMNS

(a) DETAILS OF DESIGN

In Figs. 1 to 4 are shown details of all structural sections employed in the tests. The lower 12 feet 8 inches constitute the test column proper, the upper enlarged extension, 3 feet in length, serving merely as a means for transmitting the load to the column. This head is protected by concrete as shown in Fig. 9, and, being above the ceiling line, is not directly exposed to the heat of the testing chamber.

The bottom base angles are designed to develop about one-half of the transverse strength of the column considered as a beam at ordinary temperature, and during the test are embedded in the fireproofing for the base plates of the testing machine. The top anchorage is designed to develop the full transverse strength of the column.

The test columns are provided with brackets near the top to introduce conditions affecting the application of the protective coverings similar to those obtaining in building construction.

On account of the prevailing use of the solid rolled and built-up H sections, more than one-half of the total number of steel columns used in the tests were of these types.

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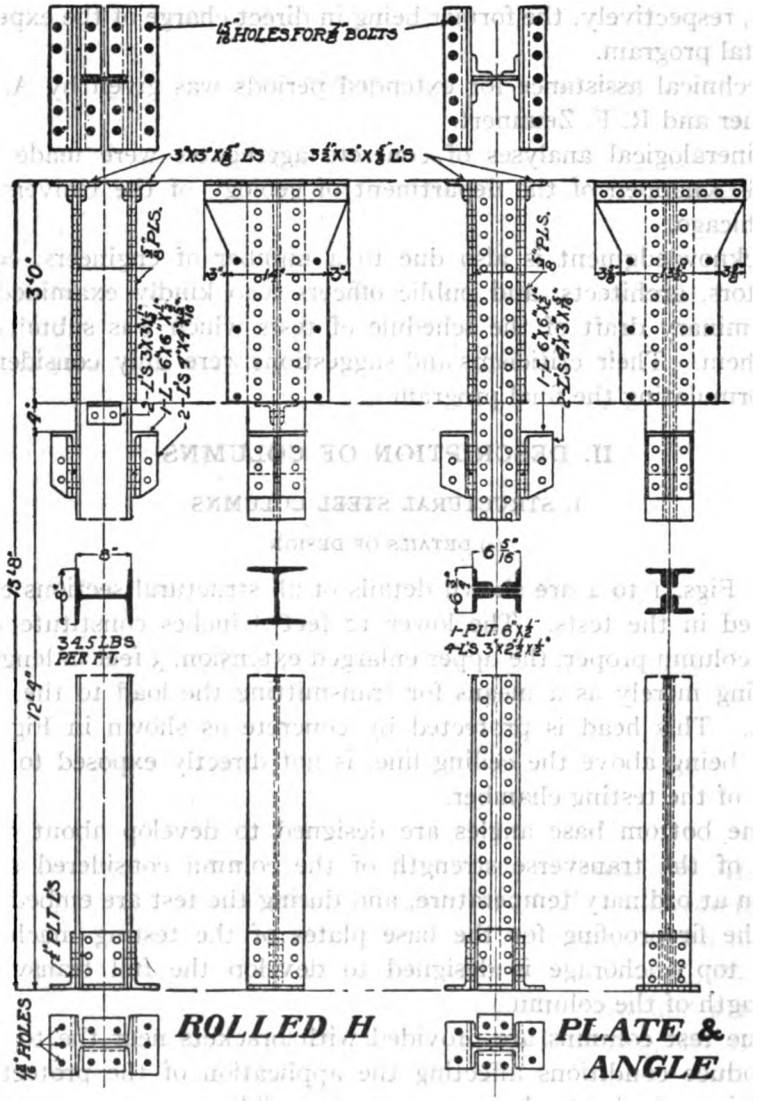

FIG. 1.—*Details of structural-steel columns. Rolled H and plate and angle sections*

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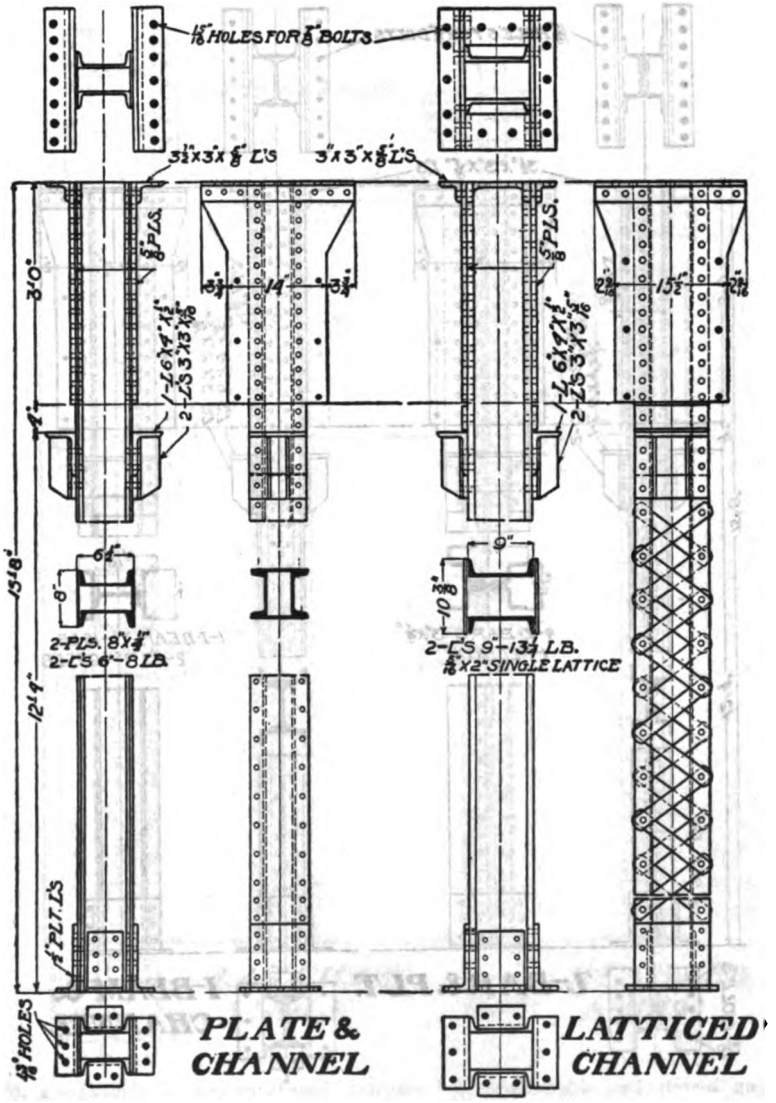

FIG. 2.—*Details of structural-steel columns. Plate and channel and latticed channel sections*

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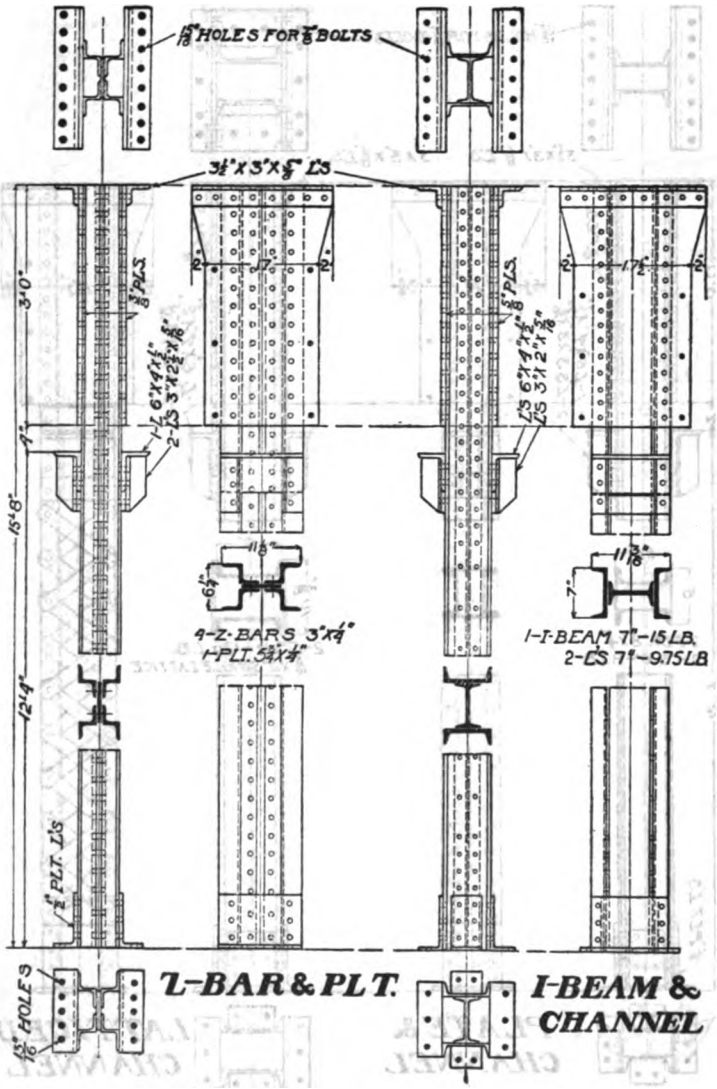

Fig. 3.—*Details of structural-steel columns. Z-bar and plate and I-beam and channel sections*

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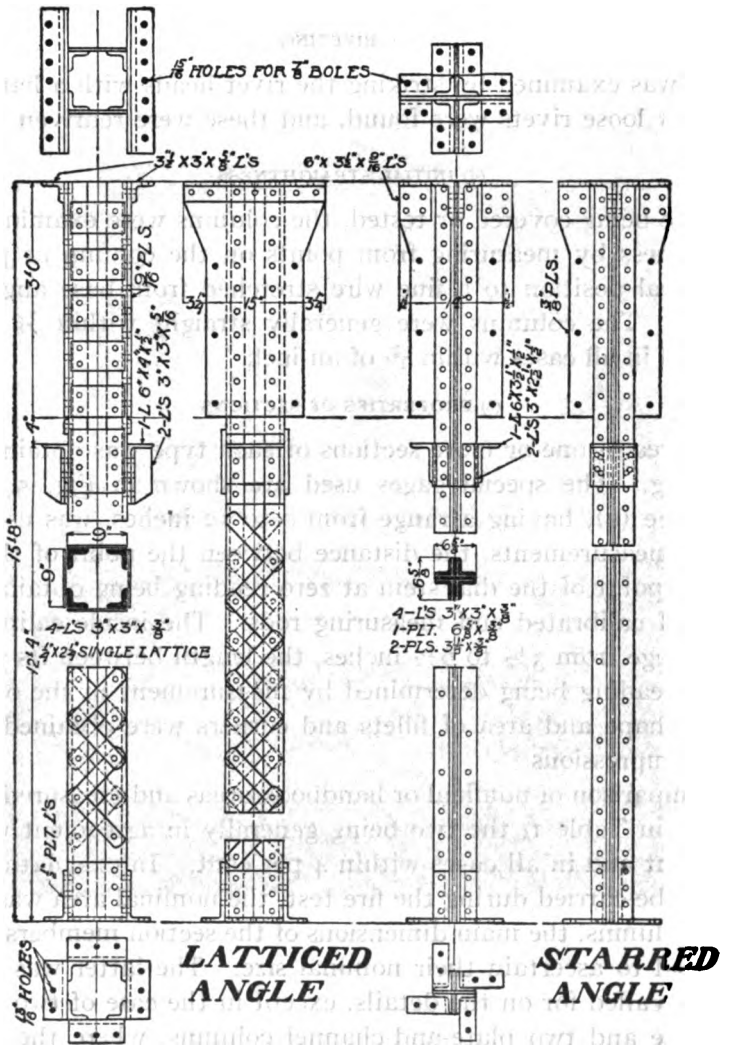

FIG. 4.—*Details of structural-steel columns. Latticed angle and starred angle sections*

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(*b*) BEARINGS

The column bases were specified to be finished by milling. While this had been done in almost all cases, it was necessary to improve most of the bearings by grinding, in order not to induce uneven stress distribution in the columns when loaded.

(*c*) RIVETING

This was examined by striking the rivet heads with a hammer. Very few loose rivets were found, and these were redriven before testing.

(*d*) INITIAL STRAIGHTNESS

Before being covered or tested, the columns were examined for straightness by measuring from points on the column as placed in vertical position to a fine wire stretched from base angles to bracket. The columns were generally straight within $\frac{1}{8}$ of an inch and in all cases within $\frac{3}{16}$ of an inch.

(*e*) PROPERTIES OF SECTIONS

The area of one or more sections of each type was obtained by calipering. The special gages used are shown in Fig. 5. The large gage (*a*), having a range from 0 to 12 inches, was used for outside measurements, the distance between the point of the pin and the point of the dial stem at zero reading being obtained by means of calibrated end measuring rods. The inside caliper (*b*) has a range from $3\frac{1}{2}$ to $6\frac{1}{2}$ inches, the length between its points at zero reading being determined by measurement in the outside gage. Shape and area of fillets and corners were obtained from plaster impressions.

A comparison of nominal or handbook areas and measured areas is given in Table 1, the two being generally in agreement within 1 per cent and in all cases within 4 per cent. In calculating the loads to be carried during the fire test, the nominal area was used for all columns, the main dimensions of the section members being measured to ascertain their nominal size. The latter was found to be as called for on the details, except in the case of two plate-and-angle and two plate-and-channel columns, where the plates were $\frac{1}{32}$ of an inch heavier than required.

Values of other essential properties of the sections are given in Table 3a (p. 33).

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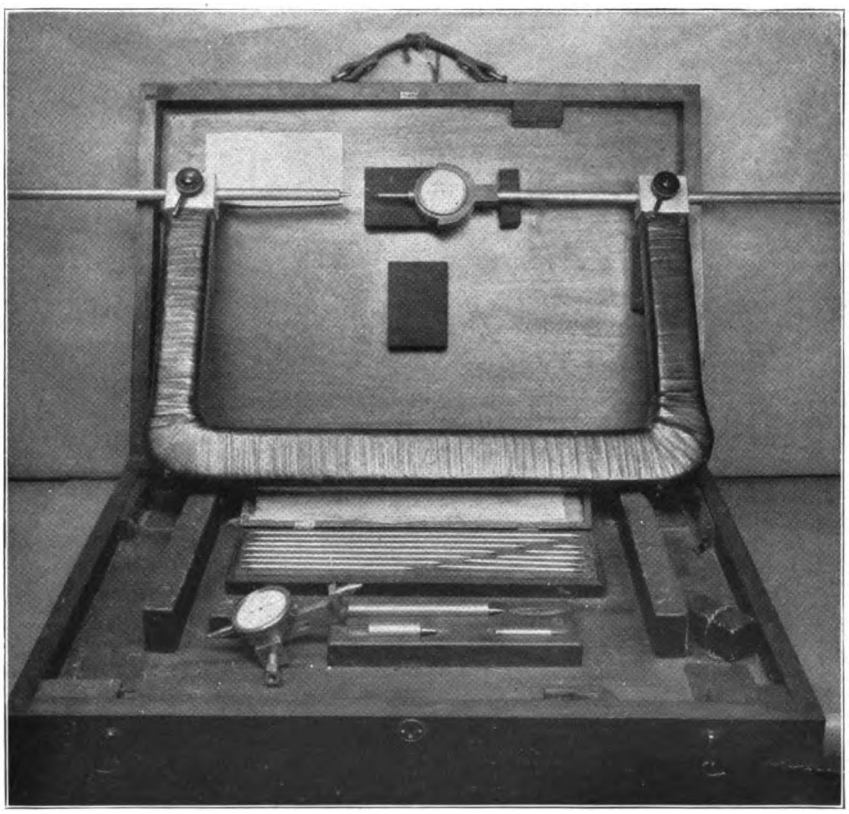

FIG. 5.—*Calipers for measuring steel shapes*

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TABLE 1.—NOMINAL AND MEASURED AREAS OF STRUCTURAL STEEL SECTIONS

Test No.	SECTION	SECTION MEMBERS	Nominal Area, Sq. In.		Measured Area, Sq. In.	
15	Rolled H	Solid Rolled H 34.51b		10 17		10.06
30	Rolled H	Solid Rolled H 34.5 lb		10.17		10.05
50	Plate and Angle	1 Plate ½ by 6 in. 4 Angles 3 by 2½ by ½ in.	3.00 10.00	13.00	2.99 9.97	12.96
51	Plate and Angle	1 Plate ½ by 6 in. 4 Angles 3 by 2½ by ½ in.	3.00 10.00	13.00	2.92 9.94	12.86
110	Plate and Angle	1 Plate ½ by 6 in. 4 Angles 3 by 2½ by ½ in.	3.00 10.00	13.00	2.99 10.21	13.20
39	Plate and Channel	2 Plates ½ by 8 in. 2 Channels 6 in.—8 lb.	4.50 4.76	9.26	4.51 4.73	9.24
107	Plate and Channel	2 Plates ½ by 8 in. 2 Channels 6 in.—8 lb.	4.00 4.76	8.76	4.19 4.57	8.76
54	Latticed Channel	2 Channels 9 in.—13¼ lb.		7.78		7.80
41	Z-bar and Plate	1 Plate ¼ by 5¾ in. 4 Z-bars 3 by ¼ in.	1.44 7.88	9.32	1.46 7.85	9.31
56	Z-bar and Plate	1 Plate ¼ by 5¾ in. 4 Z-bars 3 by ¼ in.	1.44 7.88	9.32	1.48 7.59	9.07
44	I-beam and Channel	1 I-beam 7 in.—15 lb. 2 Channels 7 in.—9¾ lb.	4.42 5.70	10.12	4.54 5.71	10.25
58	I-beam and Channel	1 I-beam 7 in.—15 lb. 2 Channels 7 in.—9¾ lb.	4.42 5.70	10.12	4.59 5.76	10.35
46	Latticed Angle	4 Angles 3 by 3 by ¾ in.		8.44		8.45

2. CAST-IRON COLUMNS

(a) DETAILS OF DESIGN AND MANUFACTURE

Structural details are given in Fig. 6. The columns shown in (*a*) and (*b*) were made by a Chicago foundry experienced in the making of building castings. They were cast horizontally, with continuous core supported by chaplets, and single gate and riser, the gate being at one end and the riser at the other.

Five of these columns were tested with ends restrained by bolting to base plates at top and bottom as shown in (*a*), and two were tested with ends not restrained, as shown in (*b*). In the latter case the bolts at the bottom were omitted, and at the top the

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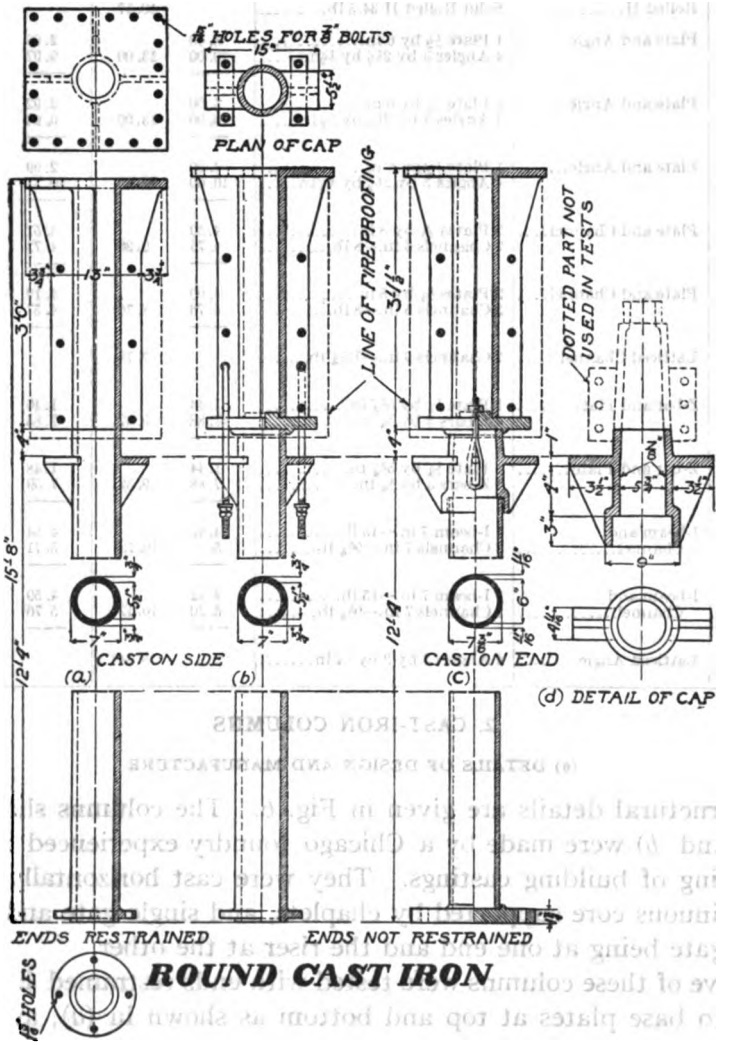

FIG. 6.—*Details of cast-iron columns*

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column was cut at the junction with the head, a bearing plate being inserted between the cut surfaces. A U bolt and strap on each side served to hold the column end in case the column should break at failure.

Three columns were of the cast-iron-pipe type with detached cap, as shown in (*c*) and (*d*) of Fig. 6. These columns were cast in vertical position. In testing, substantially the same bearing details were used as for the horizontally-cast columns that were tested with ends not restrained.

(*b*) BEARINGS

The ends of the columns were milled, the bearings being in all cases fairly even and true. All bearing surfaces of caps and bearing plates were machined. The top and bottom bearings were protected by fireproofing, the same as for the structural-steel columns, the length exposed to the fire being 12 feet.

(*c*) INITIAL STRAIGHTNESS

The amount and direction of initial curvature was determined in the same manner as for the structural-steel columns. The columns that were cast horizontally were straight within ⅛ of an inch, and those cast on end were straight within $\frac{1}{16}$ of an inch.

(*d*) VARIATIONS IN METAL THICKNESS

In the case of the horizontally cast columns, the core at the midpoint of the length was found to have been displaced by amounts varying from $\frac{1}{16}$ to ¼ of an inch for the individual columns. The minimum thickness of metal was ½ an inch against a nominal thickness of ¾ of an inch. The area of metal exceeded the nominal area by amounts up to about 1 square inch. The exact effective area was difficult to determine on account of the roughness of the interior surface.

For the vertically cast columns the thickness varied from ⅝ to ¾ of an inch against a nominal thickness of $\frac{11}{16}$ of an inch, the area being generally about ½ square inch in excess of the nominal.

3. PIPE COLUMNS

(*a*) DETAILS OF DESIGN AND MANUFACTURE

Two concrete-filled pipe columns were tested, one made with a standard 7-inch pipe, and the other with a standard 8-inch pipe reinforced in the fill with four 3½ by 3½ by ⅜ inch angles riveted back to back. Details of columns and bearings are given in Fig. 7. The details at the top are similar to those of the

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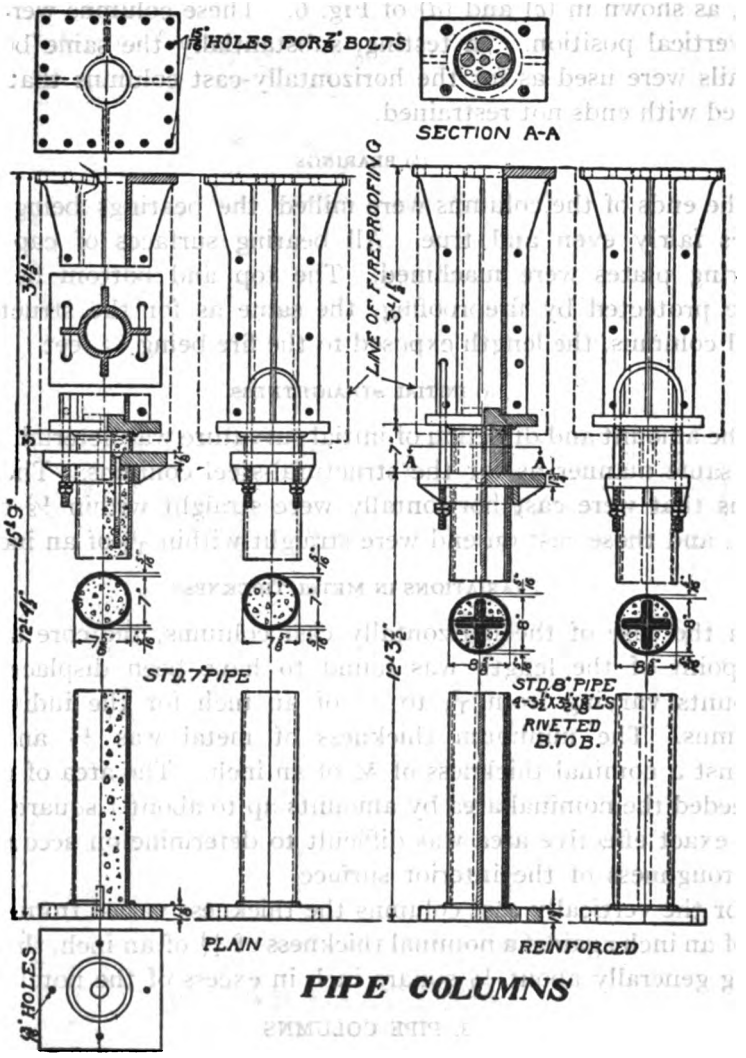

FIG. 7.—*Details of pipe columns*

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cast-iron columns that were tested with unrestrained ends except that a short strut was placed between the top of the cap and the upper bearing plate to obtain conditions more nearly representative of use in buildings.

The 7-inch pipe column was filled at the manufacturer's plant especially for the test and specimens of concrete and concrete aggregates were secured. The concrete mixture was 1 part Portland cement, 1½ parts Cambridge (Mass.) bank sand, and 3 parts crushed blue trap rock quarried at Westfield, Mass. The reinforced-pipe column was obtained from a stock of completed columns, and specimens of the concrete aggregates could not be obtained, but they were said to be the same as for the 7-inch column.

(b) BEARINGS

The bearings were square and unrestrained. The bearing surfaces of the caps were unfinished; base and top bearing plates were machined. The pipes had sawed ends that were fairly even. The concrete on the bottom bearing of the 7-inch pipe column projected about $\frac{1}{16}$ of an inch below the pipe.

4. REINFORCED CONCRETE COLUMNS

(a) DETAILS OF DESIGN

The types tested include square and round longitudinally reinforced columns, and round columns with lateral reinforcement of spiral hooping and longitudinal bar reinforcement. Details are shown in Figs. 8 and 9. The spiral reinforcement constitutes in volume about 1 per cent of the contained concrete. The size and spacing of the lateral ties in the longitudinally reinforced columns represent current practice with respect to this detail.

The concrete was of 1 : 2 : 4 mixture. In the columns for the fire tests, Fox River (Ill.) sand with Chicago limestone and Long Island (N. Y.) sand with New York trap rock were the two combinations of aggregates used. For the fire and water tests each column was cast in three 4-foot sections with concrete of different aggregates in each section.

(b) BEARINGS AND END RESTRAINT

The lower ends of the vertical bars were ground true and abutted on the base plate on which the column was cast, the upper ends of the bars terminating ½ an inch below the top bearing plate. The head was cast monolithic with the test column proper, and was

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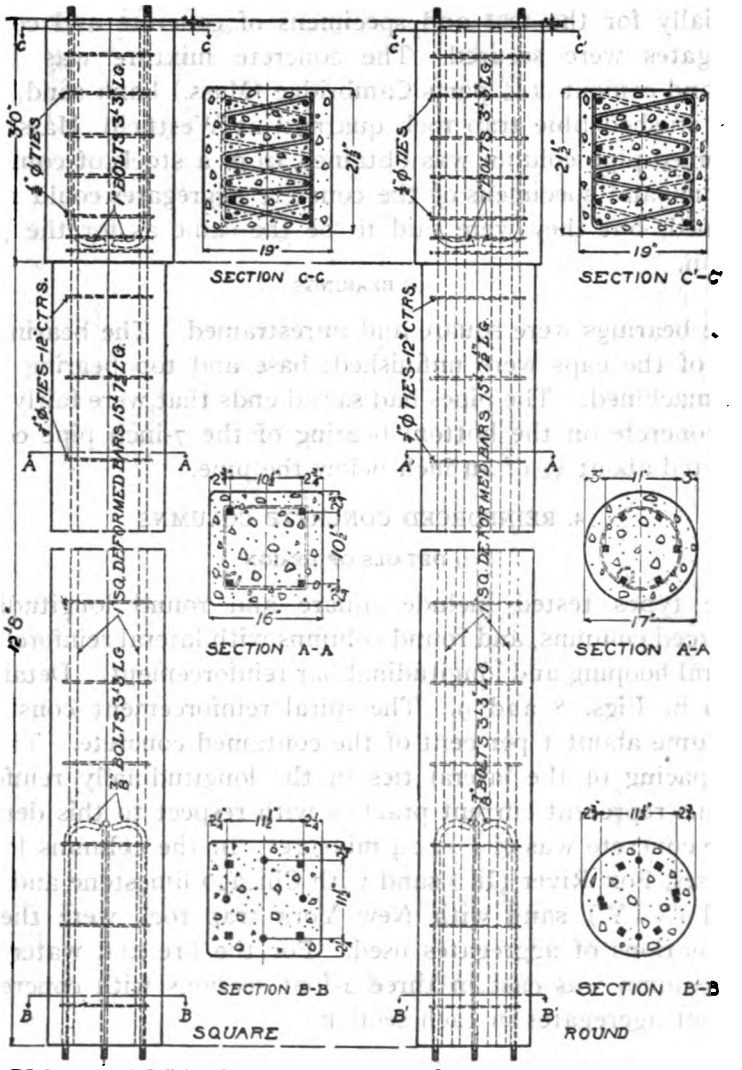

FIG. 8.—*Details of vertically reinforced concrete columns*

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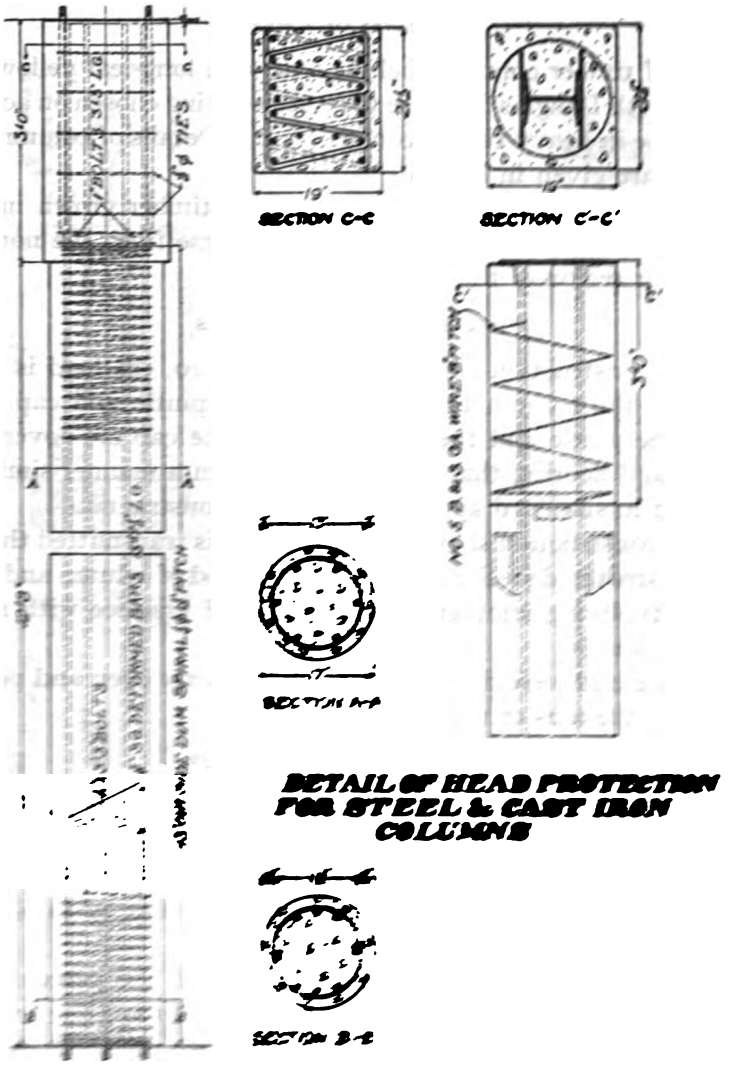

FIG. [illegible]

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suitably reinforced and anchored into the top bearing plate. The latter was set in Portland cement mortar. At the bottom, the the column was tied to the base plate with four ⅞-inch bolts.

5. TIMBER COLUMNS

(a) SPECIES OF TIMBER

Tests of timber columns include four with long-leaf yellow pine and two with Douglas fir, these two species being chosen on account of wide use in heavy timber construction. Details of columns and bearings are given in Fig. 10.

The yellow-pine columns were cut from timber grown in Pike County, Miss. The Douglas-fir columns came from the northern Douglas-fir region of the Pacific coast.

(b) CAP AND BEARING DETAILS

In the construction shown at (*a*) in Fig. 10, the load is transmitted to the column through a cast-iron pintle and cap. The side of the pintle and the top bearing of the cap are covered by timbers and flooring that frame into them in a manner similar to that used in standard applications of mill construction.

In the construction shown at (*b*) the load is transmitted through a timber strut and steel-plate cap, this method of column and beam support typifying another form of standard practice with regard to these details.

The bearings were finished so as to be fairly even and perpendicular to the axis of the column.

(c) PROPERTIES OF THE TIMBER

The chief characteristics of the timber are given in Table 2. The columns were select structural material with few knots or other defects. They were surfaced on the sides and the corners were slightly beveled. A nominal section of 11⅜ by 11⅜ inches was assumed in calculating the working load.

The number of annual rings per inch and the percentage of summer wood were determined on a representative line over the third, fourth, and fifth inches from the pith, the values given being the average for the two end faces. The resin content was obtained by extracting borings from representative points in the section with benzole and drying to constant weight at 70° C.

The moisture content and dry weight were determined from disks 1 inch thick cut 2 feet and 4 feet from the end of a timber of the same size as the test columns and which had been subjected to

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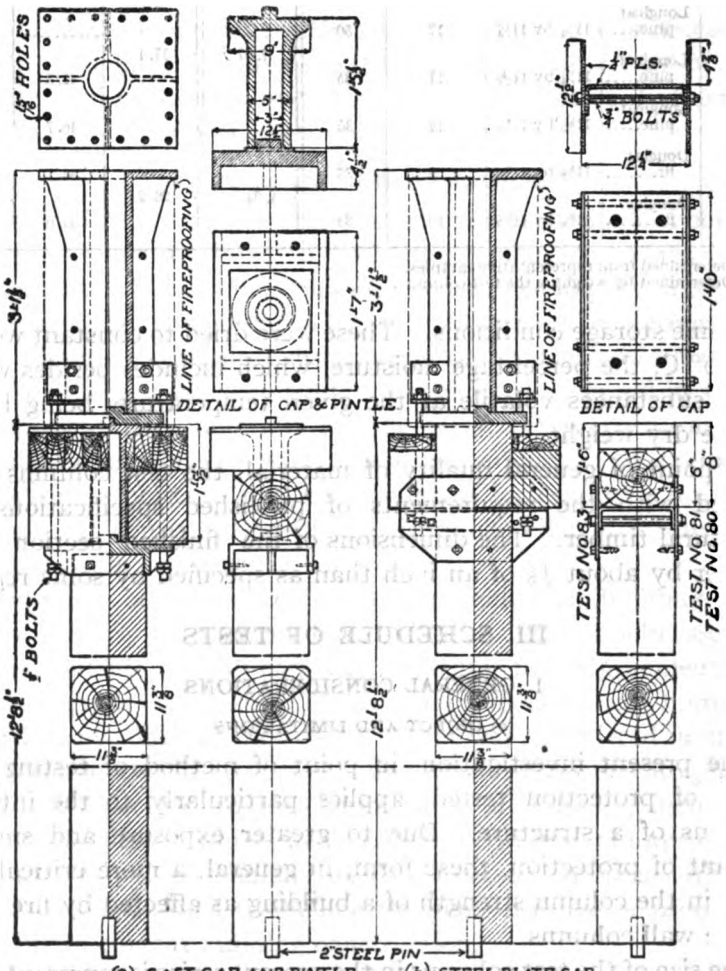

TIMBER COLUMNS

FIG. 10.—*Details of timber columns*

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TABLE 2.—PROPERTIES OF TIMBER IN TEST COLUMNS

Test No.	Species	Dimension of Section, In.	Number of Rings per In.	Summerwood, Percent	*Rosin Content, Percent	*Moisture Content, Percent	Weight per Cu. Ft., Pounds	
							**As Tested	*Oven Dry
78	Longleaf pine.....	11 1/16 by 11 1/16	14	35	7.07	17.1	41.3	34.7
79	Longleaf pine.....	11 1/16 by 11 1/16	17	50			.....	
80	Longleaf pine.....	11 1/16 by 11 1/16	11	35			41.3	
81	Longleaf pine.....	11 1/16 by 11 1/16	14	35			40.7	
82	Douglas fir.....	11 1/16 by 11 1/16	9	33	1.34	26.2	36.1	30.1
83	Douglas fir.....	11 1/16 by 11 1/16	10	33			39.0	

*Determined from representative samples.
**Determined by weighing the test columns.

the same storage conditions. These were dried to constant weight at 100° C, the percentage moisture, which includes besides water other substances volatile at the given temperature, being based on the dry weight.

In point of general quality of material, the test columns conformed with the requirements of published specifications for structural timber. The dimensions of the finished section were smaller by about 1/8 of an inch than as specified by some regulations.

III. SCHEDULE OF TESTS

1. GENERAL CONSIDERATIONS

(a) OBJECT AND LIMITATIONS

The present investigation, in point of method of testing and types of protection tested, applies particularly to the interior columns of a structure. Due to greater exposure and smaller amount of protection, these form, in general, a more critical element in the column strength of a building as affected by fire than do the wall columns.

The size of the test columns in the present series is representative of those present in buildings of moderate height and of those under upper floors in higher buildings. The sizes involved were deemed to be best adapted for the initial investigation of the many variables pertaining to the fire resistance of building columns as designed and protected according to methods of modern building practice. Subsequent investigation of columns designed for the

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heavier loads should be materially simplified by results of tests with columns of the proportions chosen for the present series.

The number of variables presented by the variety in material and form of columns, material and amount of protection, as well as the different methods of application, is greater than can be fully covered by a single series of tests, considering the limitations in time and expense incident with such an effort. It was necessary, therefore, to limit the investigation to the main forms of construction and protection, and in amount of protection, to the minimum and maximum as generally used.

(b) PRELIMINARY WORK ON SCHEDULE

This consisted of (1) investigation of existing methods of column design and column protection, including comparative study of the requirements of municipal building codes; (2) inquiry into field methods of erection and placing of columns and coverings; (3) study of the geographical distribution of production and use of materials for protective coverings of each type as an aid in selecting representative materials for the tests; (4) preparation of a preliminary schedule of tests which was submitted to engineers, contractors, architects, and public officers for criticisms and suggestions; (5) consideration of criticisms offered and formulation of a final testing program.

In amount of protection for interior columns, building ordinances require in general from 2 to 4 inch thicknesses of material outside of load-carrying elements, the requirements for the same grade of construction varying between these limits, as prescribed by different city codes. As it was considered desirable to determine the fire-resisting value of constructions in general use, it was decided to test protections of 2 and 4 inch commercial thicknesses, which, in connection with tests of unprotected and partly protected columns, as also of plaster on metal-lath protections, were deemed to include the general range of protection occurring in current building construction.

The schedule, as finally adopted at the conference of representatives of the cooperating units held in March, 1916, embodied many suggestions received in criticism of the preliminary schedule and was considered by all concerned to be the best procedure practicable, considering the extent of the field and the number of tests to be made.

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2. SCHEDULE OF FIRE TESTS

(a) UNPROTECTED COLUMNS

Details of design of structural-steel, cast-iron, and pipe columns are shown in Figs. 1, 2, 3, 4, 6, and 7, and the calculated and applied working loads pertaining to each section are given in Table 41.

The unprotected structural-steel columns, which comprise one of each section type, are scheduled in Table 3*a*, together with the chief properties of their sections. These hold for the lower 12 feet, 8 inches of their length.

The list of unprotected cast-iron and pipe columns is given in Table 3*b*. Of the unprotected cast-iron columns, one was tested with ends restrained and three with unrestrained ends. Of the latter, one column, No. 11, was filled with concrete to increase its fire resistance. They were all horizontally cast, except No. 10A, which was a cast-iron pipe column.

The tests with pipe columns comprise one with a standard 7-inch pipe having plain concrete fill and one with an 8-inch pipe reinforced in the fill with structural angles.

(b) PARTLY PROTECTED COLUMNS

These include nine structural steel columns partly protected by filling the reentrant portions or interior with concrete and are scheduled in Table 3*c*, where also are shown the detail sections.

The proportions of the concrete mixtures used throughout this investigation are based on volume parts of the respective materials, Portland cement, sand, and coarse aggregate, taken in the given order. One bag of Portland cement weighing 94 pounds net was taken to be 1 cubic foot. The sand and coarse aggregate were measured by volume in the condition of density incident with shoveling them into the measures. The concrete sands as used had an average moisture content of 3 per cent. The Portland cement used in all tests of the whole series, except in those with pipe columns, was supplied from a mill in the Chicago district.

(c) COLUMNS PROTECTED BY PLASTER ON METAL LATH

Details of protection are given in Table 3*d*. The metal lath in Nos. 23, 25, and 26 was No. 24 expanded metal weighing 3.4 pounds per square yard including paint. That for test No. 24 was of 0.046-inch diameter wire, woven into ⅜-inch square mesh and weighing, painted, 3.2 pounds per square yard. The ribbed, expanded-metal lath in No. 27 had ribs ¾ inch high spaced 3½ inches apart, the weight being 7.9 pounds per square yard.

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TABLE 3a.—SCHEDULE OF FIRE TESTS
Unprotected Structural Steel Columns
l—Effective Length, 12 ft. 8 in.

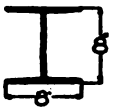
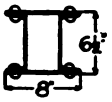
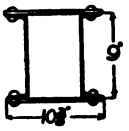
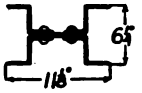
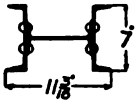
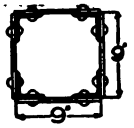
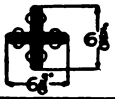
Test No.	SECTION	SECTION MEMBERS	Nominal Area, Sq. In.	Least Radius of Gyration, r, In.	$\frac{l}{r}$
1	Rolled H 	Solid Rolled H, 8 in.—34⅓ lb..	10.17	2.01	75.6
2	Plate and Angle	1 Plate, ½ by 6 in. 4 Angles, 3 by 2½ by ½ in.	13.00	1.36	111.8
3	Plate and Channel 	2 Plates, ¼ by 8 in. 2 Channels, 6 in.—8 lb.	8.76	2.35	66.7
4	Latticed Channel 	2 Channels, 9 in.—13¼ lb. Single lattice, $\frac{5}{16}$ by 2 in.	7.78	3.43	44.0
5	Z-bar and Plate 	1 Plate, ¼ by 5¾ in. 4 Z-bars, 3 by ¼ in.	9.32	1.86	81.7
6	I-beam and Channel 	1 I-beam, 7 in.—15 lb. 2 Channels, 7 in.—9¾ lb.	10.12	2.11	72.1
7	Latticed Angle 	4 Angles, 3 by 3 by ¼ in. Single lattice, ¼ by 2¼ in.	8.44	3.73	40.7
8	Starred Angle 	4 Angles, 3 by 3 by ⅜ in. 1 Plate, ⅜ by 6½ in. 2 Plates, ⅜ by 3½ in.	13.27	1.40	108.8

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TABLE 3b.—SCHEDULE OF FIRE TESTS

Unprotected Cast Iron and Pipe Columns

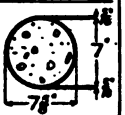
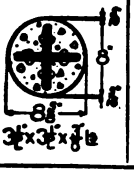
Test No.	SECTION	DETAILS	Nominal Area, Sq. In.	Effective Length, l, In.	Least Radius of Gyration, r, In.	$\frac{l}{r}$
9	Round Cast Iron, Horizontally cast	Ends restrained	14.73	152	2.23	66.2
10	Round Cast Iron, Horizontally cast	Ends not restrained	14.73	152	2.23	66.2
10-A	Round Cast Iron, Vertically cast	Ends not restrained	14.45	150¼	2.38	63.2
11	Round Cast Iron, Horizontally cast, Concrete filled	Ends not restrained Concrete, 1:2:4 Portland cement Joliet sand Joliet gravel	14.73	152	2.23	66.2
12	Steel Pipe, Concrete filled 	Ends not restrained Concrete, 1:1½:3 Portland cement Cambridge sand Westfield blue trap	Steel 6.93 Concrete 38.7	149	2.34	63.6
13	Reinforced Steel Pipe, Concrete filled 	Ends not restrained 4 angles inside pipe, 3½ in. 3½ in. ⅜ in. Concrete same as for No. 12	Steel 18.36 Concrete 40.1	152¾	2.24	68.2

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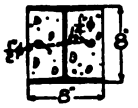
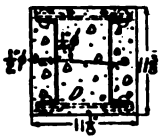
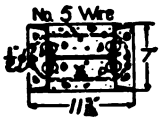
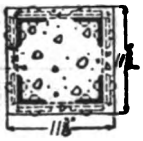
TABLE 3c.—SCHEDULE OF FIRE TESTS

Columns Partly Protected by Concrete

Test No.	SECTION	PROTECTION	
		Mixture	Material
14	Rolled H 	1:2:4	Portland cement Joliet sand Joliet gravel
15	Rolled H	1:2:4	Portland cement Plum Island sand Rockport granite
16	Plate and Angle	1:2:4	Portland cement Long Island sand New York trap
17	Plate and Angle	1:1½:4½	Portland cement Long Island sand Hard coal cinders
18	Latticed Channel 	1:2:4	Portland cement Long Island sand New York trap
19	Z-bar and Plate	1:2:5	Portland cement Fox River sand Chicago limestone
20	I-beam and Channel 	1:2:5	Portland cement Long Island sand New York trap
21	I-beam and Channel	1:2:5	Portland cement Long Island sand New York trap
22	Latticed Angle 	1:2:4	Portland cement Fox River sand Chicago limestone

NOTE: The horizontal ties, consisting of ¼-inch bars or No. 5 wire, are bent around or wired to the vertical ¾-inch bars, and are spaced 18 inches apart vertically.

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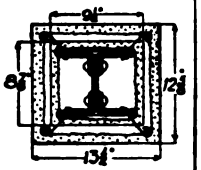
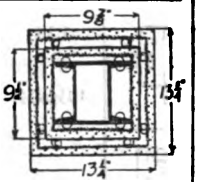
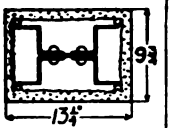
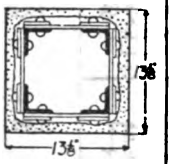
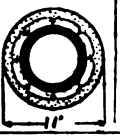
TABLE 3d.—SCHEDULE OF FIRE TESTS

Columns Protected by Plaster on Metal Lath

Test No.	SECTION	*PROTECTION
23	Plate and Angle 	Two 2-coat layers of Portland cement plaster on No. 24 expanded metal lath, each layer 1 in. thick, with a ¾-in. air space between layers
24	Plate and Channel 	Two 2-coat layers of Portland cement plaster on woven wire lath, ½-in. mesh, each layer ⅞ in. thick, with a ¾-in. air space between layers
25	Z-bar and Plate 	One 2-coat layer of Portland cement plaster, 1 in. thick, on No. 24 expanded metal lath
26	Latticed Angle 	One 2-coat layer of Portland cement plaster, 1⅛ in. thick, on No. 24 expanded metal lath
27	Round Cast Iron Column Section same as No. 9 	One 2-coat layer of Portland cement plaster, 1¼ in. thick, on high ribbed expanded metal lath with ½-in. broken air space

*The plaster consisted of 1 part Portland cement, 1/10 part hydrated lime, 2½ parts coarse lake sand. Hair was used in the first coat; the second coat was trowelled smooth.

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The proportion of the plaster was 1 part Portland cement, $\frac{1}{10}$ part hydrated lime, and 2½ parts lake sand of medium grade of coarseness, all materials being measured by loose volume. The sand had an average moisture content of 3 per cent.

The thickness of the layers was measured from the inside of the lath. For Nos. 23, 24, and 25 a layer thickness of 1 inch was desired and was attained in Nos. 23 and 25. In No. 24 the layer thickness averaged ⅞ of an inch. In No. 26 an attempt was made to place a 2-inch thick layer, but this proved impracticable with the given section using two body coats, an average thickness of 1⅛ inches being finally attained. In No. 27 an effort was made to make a solid covering by pushing the plaster through the lath against the outside of the column. This succeeded only partly, a broken air space averaging ½ inch in depth remaining between the covering and the column.

For all plaster on metal-lath protections the extreme edges of the bracket angles near the top of the column were covered by a single layer 1 inch thick.

The coverings were finished by troweling and floating the second coat to a smooth surface.

(*d*) COLUMNS PROTECTED BY CONCRETE

The tests with concrete protections are described in Table 3*e*.

Six combinations of fine and coarse concrete aggregates as used in building construction in four large industrial centers are included. The aggregate combinations and districts represented are (1) Rockport granite with Plum Island sand for the Boston (Mass.) district; (2) Chicago limestone with Fox River sand and Joliet gravel with Joliet sand for the Chicago (Ill.) district; (3) Cleveland sandstone with Pelee Island sand for the Cleveland (Ohio) district; (4) New York trap rock with Long Island sand and hard-coal cinders with Long Island sand for the New York (N. Y.) district. The districts were chosen so as to obtain representation for the main groups of rocks that are used as concrete aggregate. The aggregates are further described in paragraph 6 of Section V (p. 61).

The proportions of the mixtures used were 1 : 2 : 4 and 1 : 3 : 5 for the stone and gravel concrete, and for the cinder concrete, 1 : 1½ : 4½ and 1 : 2 : 5. The cinders were used unscreened as received, except that pieces larger than 1 inch were crushed to smaller size.

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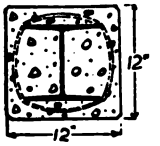
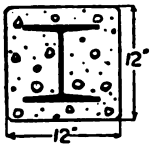
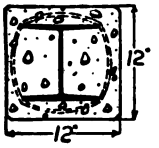
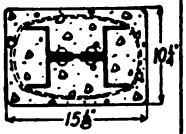
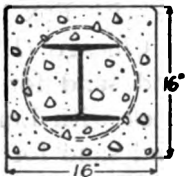
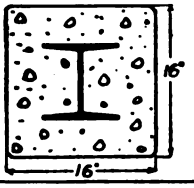
TABLE 3e.—SCHEDULE OF FIRE TESTS

Columns Protected by Concrete

Test No.	SECTION	PROTECTION		
		Thickness, In.	Mixture	Material
28	Rolled H 	2	1:2:4	Portland cement Fox River sand Chicago limestone
28 A	Rolled H 	2	1:2:4	Portland cement Fox River sand Chicago limestone
29	Rolled H	2	1:2:4	Portland cement Long Island sand New York trap
30	Rolled H 	2	1:2:4	Portland cement Joliet sand Joliet gravel
31	Rolled H	2	1:2:4	Portland cement Pelee Island sand Cleveland sandstone
32	Rolled H	2	1:2:5	Portland cement Long Island sand Hard coal cinders
32 A	Z-bar and Plate 	2	1:2:5	Portland cement Long Island sand Hard coal cinders
33	Rolled H 	4	1:2:4	Portland cement Fox River sand Chicago limestone
33 A	Rolled H 	4	1:2:4	Portland cement Fox River sand Chicago limestone

NOTE: Ties where used are of No. 5, B. & S. gage, steel wire, wound spirally on a pitch of 8 in.

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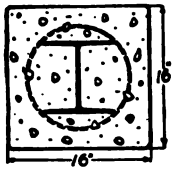
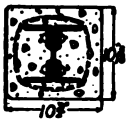
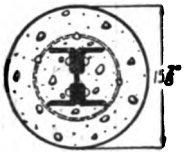
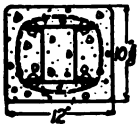
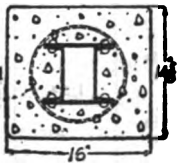
TABLE 3c.—SCHEDULE OF FIRE TESTS

Columns Protected by Concrete—Continued

Test No.	SECTION	PROTECTION		
		Thickness, In.	Mixture	Material
34	Rolled H 	4	1:2:4	Portland cement Plum Island sand Rockport granite
34A	Rolled H	4	1:2:4	Portland cement Plum Island sand Rockport granite
35	Rolled H	4	1:3:5	Portland cement Fox River sand Chicago limestone
36	Plate and Angle 	2	1:2:4	Portland cement Long Island sand New York trap
37	Plate and Angle 	4	1:2:4	Portland cement Long Island sand New York trap
38	Plate and Channel 	2	1:2:4	Portland cement Joliet sand Joliet gravel
39	Plate and Channel 	4	1:2:4	Portland cement Meramec River sand Meramec River gravel

NOTE: Wire ties are of No. 5, B. & S. gage, steel wire, wound spirally on a pitch of 8 in.

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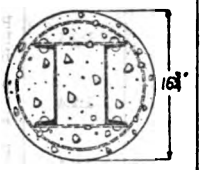
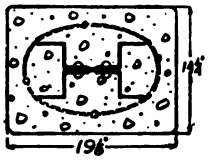
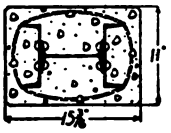
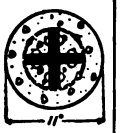
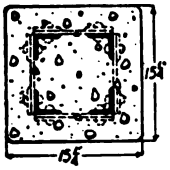
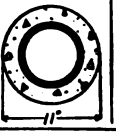
TABLE 3e.—SCHEDULE OF FIRE TESTS

Columns Protected by Concrete—Concluded

Test No.	SECTION	PROTECTION		
		Thickness, In.	Mixture	Material
40	Latticed Channel 	2	1:2:4	Portland cement Long Island sand New York trap
41	Z-bar and Plate 	4	1:3:5	Portland cement Fox River sand. Chicago limestone
42	Z-bar and Plate	4	1:3:5	Portland cement Fox River sand Chicago limestone
43	I-beam and Channel 	2	1:2:4	Portland cement Pelee Island sand Cleveland sandstone
44	I-beam and Channel	2	1:3:5	Portland cement Pelee Island sand Cleveland sandstone
45	Starred Angle 	2	1:2:4	Portland cement Meramec River sand Meramec River gravel
46	Latticed Angle 	2-in. outside rivets. 3½-in. outside angles.	1:2:4	Portland cement Long Island sand New York trap
47	Round Cast Iron 	2	1:2:5	Portland cement Long Island sand Hard coal cinders

NOTE: Wire ties where used are of No. 5, B. & S. gage, steel wire wound spirally on a pitch of 8 in.

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Ties consisting of No. 5 (B. and S. gage) bright basic steel wire were wound spirally around the structural section on a vertical pitch of 8 inches. The tie was omitted in Test No. 28A and 33A in order to determine what effect, if any, it has on the effectiveness of the covering. In No. 46 it was omitted because the latticed section was deemed to afford sufficient support for the outside covering. In No. 47 an attempt was made to place the covering with the tie wire supported on 1-inch T bars. This proved impracticable on account of the obstruction it made to the flow of the concrete, this covering being, therefore, placed without tie.

For the square coverings the thickness was measured, respectively, from the face of the flange and from its extreme edge. For the round protections, test Nos. 37 and 40, the thickness of covering on the face of the flange was greater than the given nominal thickness and that on the flange edge smaller than the given thickness, each by about $\frac{5}{8}$ of an inch.

For the concrete, as well as all other full protections, the thickness of covering on the extreme edges of the bracket angles near the top of the column was 1 inch.

(e) COLUMNS PROTECTED BY HOLLOW CLAY TILE

The tests with hollow clay-tile protections are given in Table 3*f*.

Five kinds of tile from as many producing regions were used for the coverings. These include two varieties of surface-clay tile, one of shale, and two of semifire clay. They are further described in paragraph 11 of Section V (p. 73).

The tile was set in mortar consisting of 1 part, by loose volume, of Portland cement, 1 part of lime putty, and 4 parts of fine beach or bank sand. The sand had an average moisture content of 4 per cent. The thickness of mortar between the tile and the flanges of the test columns varied between the different protections from $\frac{1}{2}$ to $1\frac{1}{8}$ inches. The thickness of horizontal joints between the tile courses averaged $\frac{3}{8}$ of an inch where no wire mesh was used and $\frac{5}{8}$ of an inch where used. The vertical mortar joints varied in thickness from $\frac{1}{4}$ to $\frac{3}{4}$ of an inch, depending on the design of the covering.

The upper tile courses were set out sufficiently to allow the extreme edges of the bracket angles to be covered by a 1-inch thickness of tile and mortar.

Two forms of mechanical ties for the tile were used. One consisted of a No. 12 (B. and S. gage) iron wire, tied around the outside of each course, and the other of strips of $\frac{3}{8}$-inch wire mesh (diameter of wire, 0.046 inch) laid in the horizontal joints and lapped at the corners.

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TABLE 31.—SCHEDULE OF FIRE TESTS

Columns Protected by Hollow Clay Tile

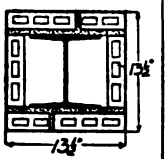
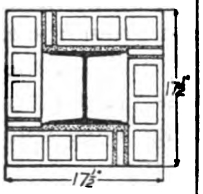
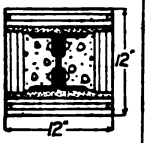
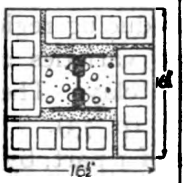
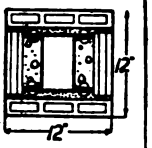
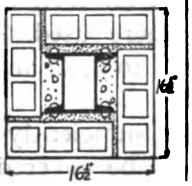
Test No.	SECTION	Thickness of Tile. In.	Kind of Tile and Method of Application	Filling
48	Rolled H 	2	Semi-fire clay, New Jersey district ¾-in mortar on flanges Outside wire ties	No filling
49	Rolled H 	4	Semi-fire clay, New Jersey district ¾-in. mortar on flanges	No filling
50	Plate and Angle 	2	Surface clay, Boston district ¼-in. mortar on flanges	1:3:5 concrete Portland cement Plum Island sand Rockport granite
50-A	Plate and Angle	2	Same as No. 50	Same as No. 50
51	Plate and Angle 	4	Surface clay, Boston district 1½-in mortar on flanges Outside wire ties	1:3:5: concrete Portland cement Plum Island sand Rockport granite
51-A	Plate and Angle	4	Same as No. 51	Same as No. 51
52	Plate and Channel 	2	Ohio shale ¾-in. mortar on flanges Outside wire ties	1:2:5 concrete Portland cement Long Island sand Hard coal cinders
53	Plate and Channel 	4	Ohio shale 1-in. mortar on flanges Outside wire ties	1:2:5: concrete Portland cement Long Island sand Hard coal cinders

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TABLE 31.—SCHEDULE OF FIRE TESTS

Columns Protected by Hollow Clay Tile—Continued

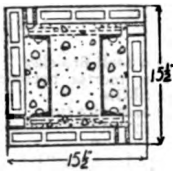
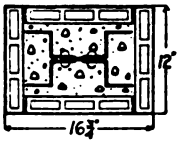
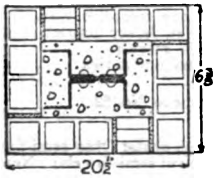
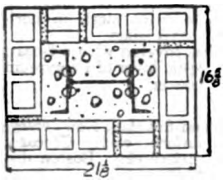
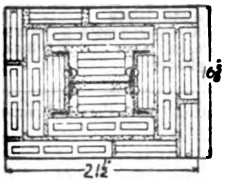
Test No.	SECTION	Thickness of Tile, In.	Kind of Tile and Method of Application	Filling
54	Latticed Channel 	2	Ohio semi-fire clay Outside wire ties	1:3:5 concrete, Portland cement Long Island sand New York trap
55	Z-bar and Plate 	2	Ohio semi-fire clay Outside wire ties	1:3:5: concrete, Portland cement Fox River sand Chicago limestone
56	Z-bar and Plate 	4	Ohio semi-fire clay ½-in. wire mesh in horizontal joints	1:3:5: concrete, Portland cement Fox River sand Chicago limestone
57	I-beam and Channel 	4	Surface clay, Chicago district Outside wire ties	1:3:5: concrete, Portland cement Fox River sand Chicago limestone
58	I-beam and Channel 	Two 2-in.	Surface clay, Chicago district ½-in. mortar on flanges ½-in. wire mesh in horizontal joints	Hollow clay tile, 2 by 12 by 6 in. at at ends 3 by 12 by 6 in. at sides
59	I-beam and Channel	Two 2-in.	Surface clay, Chicago district ½-in. mortar on flanges Outside wire ties	Hollow clay tile, 2 by 12 by 6 in. at ends 3 by 12 by 6 in. at sides

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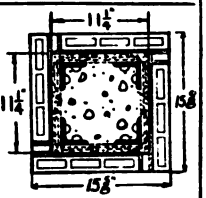
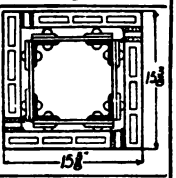
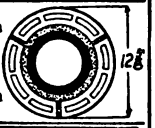
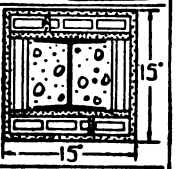
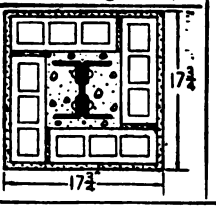
TABLE 31.—SCHEDULE OF FIRE TESTS

Columns Protected by Hollow Clay Tile—Concluded

Test No.	SECTION	Thickness of Tile, In.	Kind of Tile and Method of Application	Filling
60	Latticed Angle 	2	Ohio semi-fire clay ¼-in. mortar between fill and tile Outside wire ties	1:2:4: concrete Portland cement Long Island sand New York trap Fill placed before tile was set
61	Latticed Angle 	2	Ohio semi-fire clay Outside wire ties	No filling
62	Round Cast Iron 	2	Porous semi-fire clay, New Jersey district ¼-in. mortar Outside wire ties	No filling
63	Round Cast Iron	2	Same as No. 62	No filling
76	Rolled H 	2	Ohio shale: Ohio semi-fire clay; semi-fire clay, New Jersey district ¼-in. mortar on flanges ½-in. wire mesh in horizontal joints Tile covered with a 2-coat, ¾-in. layer of 1:3 gypsum plaster	1:3:5: concrete Portland cement Fox River sand Chicago limestone
77	Plate and Angle 	4	Semi-fire clay, New Jersey; surface clay, Chicago; surface clay, Boston district 1¼-in. mortar on flanges ½-in. wire mesh in horizontal joints Tile covered with a 2-coat, ¾-in. layer of 1:2½ lime plaster	1:3:5 concrete Portland cement Fox River sand Chicago limestone

NOTE: The mortar used in setting the tile consisted of 1 part Portland cement, 1 part lime putty and 4 parts fine beach or bank sand.

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The filling inside of the tile, where used, consisted of concrete or hollow clay tile. The concrete was placed after the tile was set, except in case of No. 60, where the fill was placed and allowed to harden before setting the tile.

All protections were tested in the unplastered condition except those of Nos. 76 and 77. The former was plastered with a two-coat layer of gypsum plaster ¾ inch thick. The first coat of about ⅝ inch thickness consisted of 1 volume part neat, fibered, calcined gypsum and 3 parts fine lake sand, and the finish coat, 1 volume part neat, unfibered, calcined gypsum and 2 parts hydrated lime. No. 77 was plastered with lime plaster ⅝ of an inch in thickness, the first coat consisting of 1 volume part slaked-lime putty and 2½ parts of fine lake sand, the finish coat being the same as for No. 76.

(*f*) COLUMNS PROTECTED BY GYPSUM BLOCK

The tests with gypsum block protections scheduled in Table 39 consist of two with 2-inch and three with 4-inch solid block. The material was supplied from two factories, one located in the middle-western section of the country and the other in the eastern section. The proportion of the mortar used for setting the block was 1 part, by volume, of neat, unfibered, calcined gypsum, and 3 parts, by volume, of fine lake sand. The latter as used had a moisture content of about 3 per cent.

In Nos. 64 and 65 the blocks were tied with No. 22 corrugated, galvanized-iron strips, ¾ of an inch wide by 6 inches long, placed in the horizontal joints and across all vertical joints, one over each joint in the 2-inch covering and two over each joint in the 4-inch covering. In Nos. 66, 67, and 67A strips of woven wire (0.046-inch wire diameter) of ⅜-inch mesh were laid in the horizontal joints over all vertical joints. The size of the strips for the 2-inch protection was 2 by 14½ inches and for the 4-inch protections, 3½ by 16½ inches, the strips being laid so the outer edges were ½ inch from the surface of the covering. The thickness of horizontal joints averaged ½ inch between the 2-inch blocks, and ¾ inch between the 4-inch blocks. The vertical joints were about ⅜ of an inch thick.

The space between the blocks and column flanges was filled with mortar as the blocks were set. The remaining space inside of the blocks was filled with gypsum block set in mortar in the

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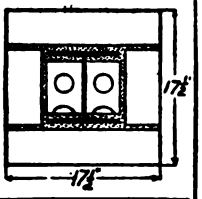
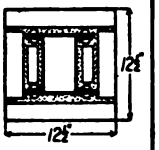
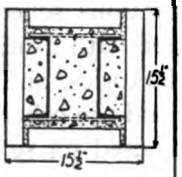
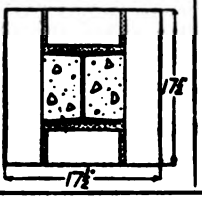
TABLE 8g.—SCHEDULE OF FIRE TESTS

Columns Protected by Gypsum Block

Test No.	SECTION	Thickness of Block, In.	Kind of Gypsum and Method of Application	Filling
64	Rolled H 	4	Western gypsum (solid) ⅝-in. mortar on flanges Corrugated wall ties in horisontal joints	Hollow Western gypsum block
65	Plate and Channel 	2	Western gypsum (solid) 1-in. mortar on flanges Corrugated wall ties in horisontal joints	Solid Western gypsum block
66	Latticed Channel 	2	Eastern gypsum (solid) ⅝-in. mortar on lattice ⅝-in. wire mesh in horisontal joints	1:1:4, Calcined gypsum Fine lake sand Broken gypsum block
67	Rolled H 	4	Eastern gypsum (solid) ⅝-in. mortar on flanges ⅝-in. wire mesh in horisontal joints	1:1:4, Calcined gypsum Fine lake sand Broken gypsum block
67-A	Rolled H	4	Same as No. 67	Same as No. 67

NOTE: The gypsum blocks were set in mortar consisting of 1 part by volume of neat unfibered calcined gypsum and 3 parts fine lake sand.

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case of Nos. 64 and 65. For the other gypsum protections the filling material consisted of 1 part, by volume, of neat, unfibered, calcined gypsum, 1 part fine lake sand, and 4 parts gypsum block broken to maximum size of 2 inches, the whole being mixed to wet consistency.

The methods of tying the blocks and filling within them conform with the recommendations of the manufacturers by whom they were supplied.

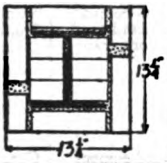
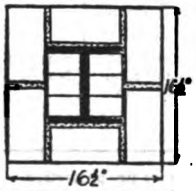
(g) COLUMNS PROTECTED BY BRICK

The two tests with brick protection are described in Table 3*h*.

The proportion of the mortar was the same as that used for the clay-tile protections. In placing the brick no metal ties were used, the brick being in each case set so as to obtain the best bond possible with the given design of covering. The thickness of the horizontal joints averaged ½ inch and that of the vertical joints varied from ¼ to 1¼ inches.

TABLE 3h.—SCHEDULE OF FIRE TESTS

Columns Protected by Brick

Test No.	SECTION	Thickness of Brick, In.	Kind of Brick	Filling
68	Rolled H 	2¼	Chicago common brick set on edge and end ½-in. mortar on flanges	Chicago common brick
69	Rolled H 	2¼	Chicago common brick laid flat ½-in. mortar on flanges	Chicago common brick

NOTE: The mortar consisted of 1 part Portland cement, 1 part lime putty and 4 parts bank sand.

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(*h*) REINFORCED-CONCRETE COLUMNS

These are scheduled in Table 3*i* and details of design are given in Figs. 8 and 9 (p. 26-27).

A thickness of 2 inches of the concrete next to the surface of the column is taken as a protective covering for the concrete and steel within it and is not included in the given effective areas.

TABLE 3i.—SCHEDULE OF FIRE TESTS

Reinforced Concrete Columns

Effective Length, 12 ft. 8 in.

Test No.	SECTION	Mix	Material	*Effective Area, Sq. In.	Reinforcement, Percent of Effective Area
70	Square Vertically Reinforced	1:2:4	Portland cement Fox River sand Chicago limestone	Concrete, 140 Steel, 4.00	Vertical, 2.78 Lateral, 0.14
71	Square Vertically Reinforced	1:2:4	Portland cement Long Island sand New York trap	Same as No. 70	Same as No. 70
72	Round Vertically Reinforced	1:2:4	Portland cement Fox River sand Chicago limestone	Concrete, 127 Steel, 6.00	Vertical, 4.52 Lateral, 0.18
73	Round Vertically Reinforced	1:2:4	Portland cement Long Island sand New York trap	Same as No. 72	Same as No. 72
74	Hooped Reinforced	1:2:4	Portland cement Fox River sand Chicago limestone	Concrete, 129 Steel, 3.38	Vertical, 2.54 Lateral, 0.99
75	Hooped Reinforced	1:2:4	Portland cement Long Island sand New York trap	Same as No. 74	Same as No. 74

*A depth of 2 in. all around deducted from the gross section for fire protection.

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(f) TIMBER COLUMNS

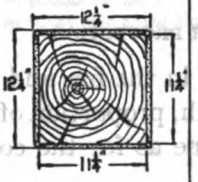
The schedule of timber columns is given in Table 3*j* and details of columns and bearings are shown in Fig. 10. The tests include two species of timber, each tested with two types of bearing details.

No. 78 was protected by a single layer of plaster on metal lath, the details of application being the same as for the protections listed in Table 3*d*. No. 80 was covered with gypsum wall board 3/8 inch thick, nailed to the timber at the corners with No. 4 lathing nails on 2-inch centers, and finished with kalsomine. The other timber columns were tested unprotected.

TABLE 3j.—SCHEDULE OF FIRE TESTS

Timber Columns

Effective Length, 12 ft. 8½ in.

Test No.	SPECIES	SECTION	Protection of Column and Cap	Bearing Details
78	Longleaf pine		One 2-coat layer of Portland cement plaster, 1 in. thick, on woven wire lath, ⅝-in. mesh, ¾-in. air space. Plaster consisted of 1 part Portland cement, 1/10 part hydrated lime, 2½ parts coarse lake sand	Cast iron cap and pintle
79	Longleaf pine		Unprotected	Cast iron cap and pintle
80	Longleaf pine		One thickness of ⅜-in. gypsum wall board with metal corner beads, nailed to column	Steel plate cap and 8 in. long timber strut.
81	Longleaf pine		Unprotected	Steel plate cap and 6 in long timber strut
82	Douglas fir		Unprotected	Cast iron cap and pintle
83	Douglas fir		Unprotected	Steel plate cap and 18 in. long timber strut

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3. SCHEDULE OF FIRE AND WATER TESTS

This series was introduced in order to determine the effect on coverings and columns of the impact and sudden cooling produced by hose streams applied to them when in a highly heated condition.

In order to introduce all of the materials used in the fire-test series with a minimum number of tests, two or three kinds of material of a given class were applied in each test.

(*a*) COLUMNS PROTECTED BY CONCRETE

The fire and water tests with concrete protections are scheduled in Table 4*a*. Three kinds of concrete were applied to each column. In this and succeeding tables the concrete is distinguished by the name of the coarse aggregate, the sand used with each being the same as in the combinations given above in paragraph 2*d* (p. 37).

The metal tie in the covering is the same as for the corresponding protections in the fire-test series. In No. 102 the tie is omitted.

(*b*) COLUMNS PROTECTED BY HOLLOW CLAY TILE

These coverings are also placed in three sections (Table 4*b*) with one of the varieties of tile used in the protections of the fire-test series in each section. The proportions of the mortar and size of metal ties are the same as for the corresponding protections of the fire-test series.

(*c*) COLUMNS PROTECTED BY GYPSUM BLOCK

The two kinds of gypsum with the method of application peculiar to each, described in paragraph 2*f* of this section are employed in the two tests (Table 4*c*). The filling for both consists of a 1 : 1 : 4 mixture by volume parts of calcined gypsum, fine lake sand, and broken gypsum block, mixed to wet consistency.

(*d*) PLASTER ON METAL-LATH PROTECTION

The protection of this type included in the fire and water series is described in Table 4*d*. The metal lath, proportion of the plaster, and method of application are the same as for the corresponding protections of the fire-test series.

(*e*) REINFORCED-CONCRETE COLUMNS

One column of each type is included in this series. Details of concrete and reinforcement are given in Table 4*e*.

(*f*) UNPROTECTED CAST-IRON COLUMNS

Two duplicate columns are listed in Table 4*f*. They are vertically cast and have bearing details as shown in Fig. 6 (*c*) (p. 22).

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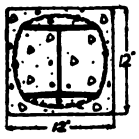
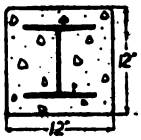
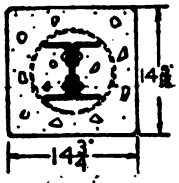
TABLE 4a.—SCHEDULE OF FIRE AND WATER TESTS

Columns Protected by Concrete

Test No.	SECTION	PROTECTION		
		Thickness, In.	Mixture	*Kind of Concrete
101	Rolled H 	2	1:2:4	Chicago limestone New York trap Joliet gravel
102	Rolled H 	2	1:2:4	New York trap Joliet gravel Chicago limestone
103	Plate and Angle 	4	1:2:4	New York trap New England granite Chicago limestone
104	Plate and Angle	2	1:2:5 1:2:4 1:2:4	Hard coal cinders Cleveland sandstone New York trap

*Three kinds of concrete were used on each column, placed in three vertical sections in the order named, from top to bottom of column.

NOTE: Ties where used are of No. 5, B. & S. gage, steel wire, wound spirally on a pitch of 8 in.

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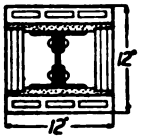
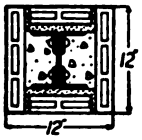
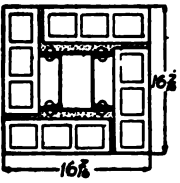
TABLE 4b.—SCHEDULE OF FIRE AND WATER TESTS

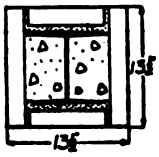
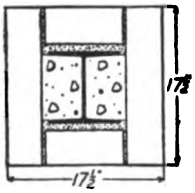
Columns Protected by Hollow Clay Tile

Test No.	SECTION	Thickness of Tile, In.	*Kind of Tile and Method of Application	Filling
105	Plate and Angle 	2	Surface clay, Boston district; semi-fire clay, N. J. district; Ohio shale ½-in. mortar on flanges Outside wire ties	No filling
106	Plate and Angle 	2	Ohio semi-fire clay; surface clay, Chicago district; Ohio semi-fire clay ½-in. mortar on flanges Outside wire ties on upper half ½-in. wire mesh in horizontal joints in lower half	1:3:5 concrete. Portland cement Long Island sand Chicago limestone
107	Plate and Channel 	4	Ohio shale; semi-fire clay, N. J. district; surface clay, Boston district 1-in. mortar on flanges Outside wire ties on upper half ½-in. wire mesh in horizontal joints in lower half	No filling

*Three kinds of tile were used on each column, placed in three vertical sections in the order named from top to bottom of column.

TABLE 4c.—SCHEDULE OF FIRE AND WATER TESTS

Columns Protected by Gypsum Block

Test No.	SECTION	Thickness of Block, In.	*Kind of Gypsum and Method of Application	Filling
108	Rolled H 	2	Western gypsum (solid); Eastern gypsum (solid) ¾-in. mortar on flanges Corrugated wall ties in horizontal joints in upper half ½-in. wire mesh in horizontal joints in lower half	1:1:4 Calcined gypsum Fine lake sand Broken gypsum block
109	Rolled H 	4	Eastern gypsum (solid); Western gypsum (solid) ¾-in. mortar on flanges ½-in. wire mesh in horizontal joints in upper half. Corrugated wall ties in horizontal joints in lower half	1:1:4 Calcined gypsum Fine lake sand Broken gypsum block

*Two kinds of block were used on each column, placed in two vertical sections in the order named from top to bottom.

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TABLE 4d.—SCHEDULE OF FIRE AND WATER TESTS

Column Protected by Plaster on Metal Lath

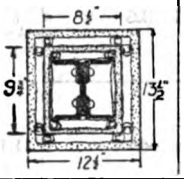
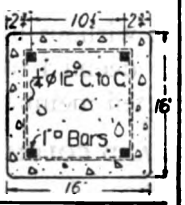
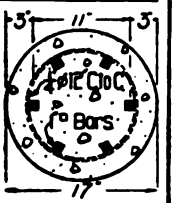
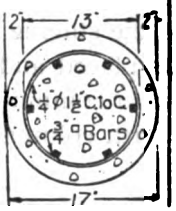
Test No.	SECTION	PROTECTION
110	Plate and Angle 	Two 2-coat layers of Portland cement plaster; inner layer ⅛ in. thick, on woven wire lath; outer layer 1 in. thick, on expanded metal lath. ¾-in. air space between layers. Proportion of plaster, 1 part Portland cement, 1/10 part hydrated lime, 2½ parts coarse lake sand

TABLE 4e.—SCHEDULE OF FIRE AND WATER TESTS

Reinforced Concrete Columns

Effective Length, 12 ft. 8 in.

Test No.	SECTION	Mixture	*Kind of Concrete	**Effective Area, Sq. In.	Reinforcement, Percent of Effective Area
111	Square Vertically Reinforced 	1:2:4	Chicago limestone Meramec R. gravel Chicago limestone	Concrete, 140 Steel, 4.00	Vertical, 2.78 Lateral, 0.14
112	Round Vertically Reinforced 	1:2:4	Chicago limestone Meramec R. gravel Joliet gravel	Concrete, 127 Steel, 6.00	Vertical, 4.52 Lateral, 0.13
113	Hooped Reinforced 	1:2:4	New York trap Meramec R. gravel Rockport granite	Concrete, 129 Steel, 3.38	Vertical, 2.54 Lateral, 0.99

*Three kinds of concrete were used in each column, placed in three sections in the order named, from top to bottom of column.

**A depth of 2 in. all around deducted from the gross section for fire protection.

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TABLE 4f.—SCHEDULE OF FIRE AND WATER TESTS

Unprotected Cast Iron Columns

Effective Length, 12 ft. 6¼ in.

Test No.	SECTION	DETAILS	Nominal Area, Sq. In.	Least Radius of Gyration, r In.	$\frac{l}{r}$
114	Round Cast Iron, Vertically cast	Ends not restrained	14.45	2.38	63.2
115	Round Cast Iron, Vertically cast	Ends not restrained	14.45	2.38	63.2

IV. PLACING OF COVERINGS AND CONCRETE COLUMNS

The work was planned so as to reproduce as nearly as possible the conditions obtaining in building construction in point of methods of application and workmanship. This was done to make the results of the tests applicable without undue allowance for differences that otherwise might be deemed to exist between the test sample and a similarly constructed member in a building.

1. CONCRETE PROTECTIONS AND COLUMNS

(*a*) FORMS AND STAGING

The wood forms were made of 1⅝-inch yellow-pine planks, with clamps spaced about 2 feet apart vertically. The round columns and coverings were cast in metal forms made of No. 12 (0.1094 inch thick) sheet steel, the forms being made into halves which were bolted together through angles riveted on their edges.

The form was supported within a staging extending to the top of the test column proper, the floor of the staging being used as a platform from which the concrete was placed and on which subsequently the form for the column head or column-head protection was erected. A view of the staging with column forms in place is shown in Fig. 11.

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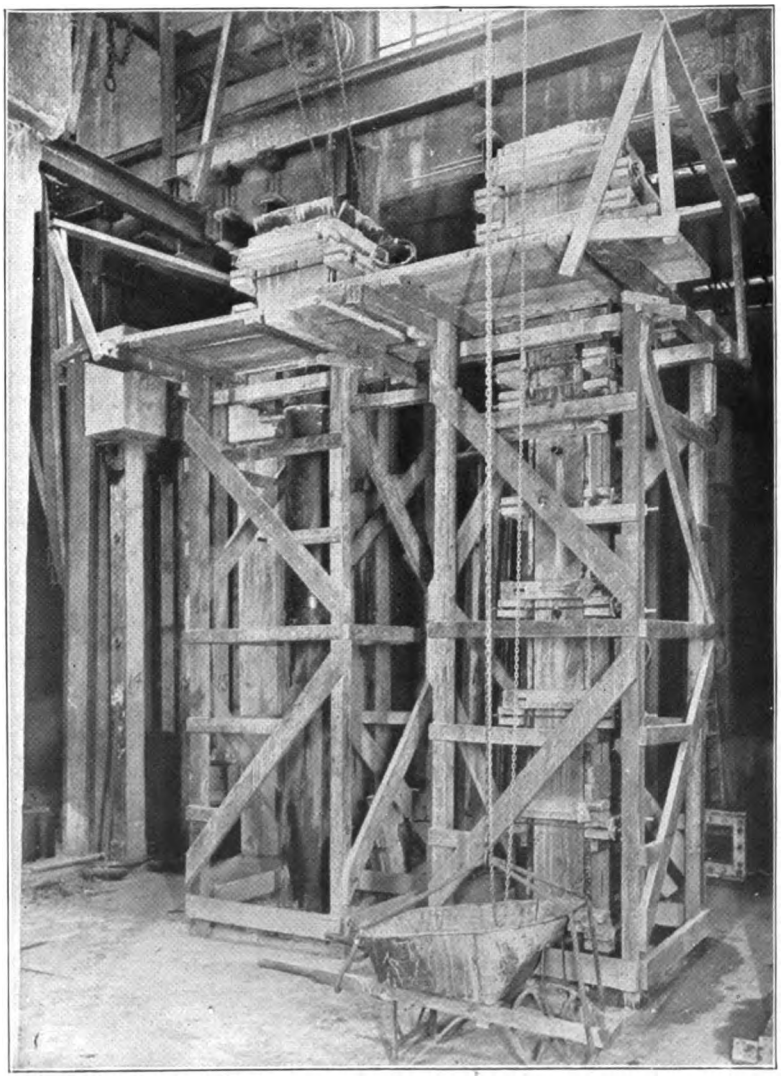

FIG. 11.—*Forms and staging for placing concrete*

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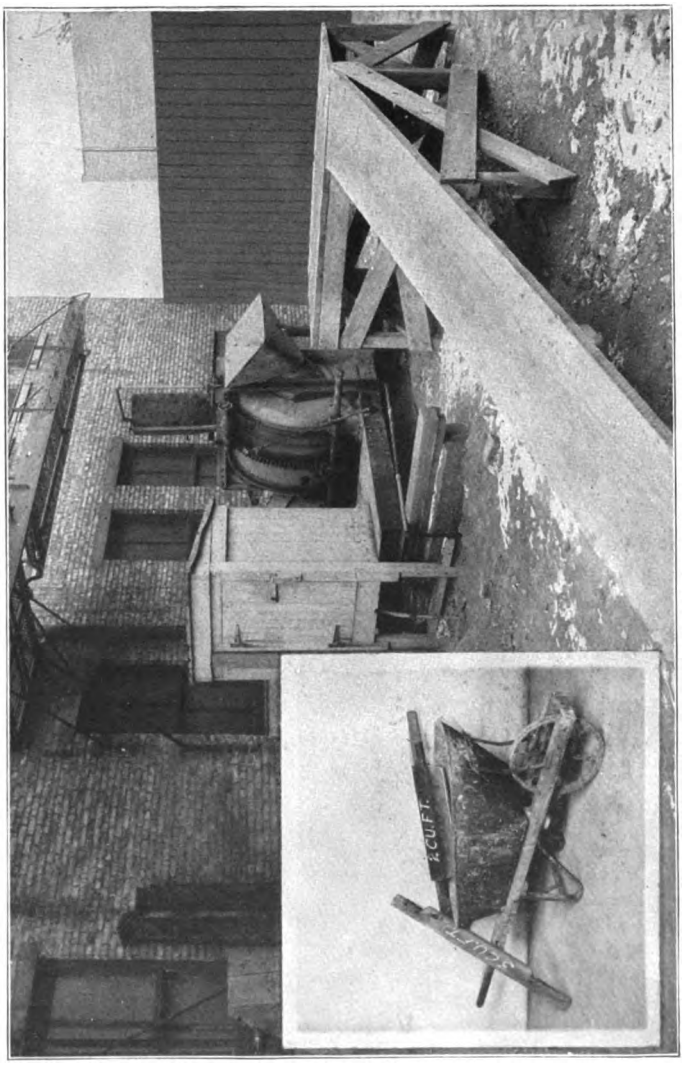

FIG. 12.—*Concrete mixer*

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(b) METHOD OF PROPORTIONING

The proportions were based on volume parts of the materials as taken from the bins, except that the Portland cement was measured in the original package, one bag, containing 94 pounds of cement, being taken to be 1 cubic foot. The sand and stone were measured in deep, steel wheelbarrows in 2 and 3 cubic-foot portions, the volume for each being determined by striking off the top with a board cut to the required shape (Fig. 12). In some tests, where the mixtures appeared lean in sand, two or three shovelfuls of sand were substituted for an equal amount of stone.

(c) MIXING AND PLACING

The concrete was mixed in a motor-driven batch mixer having a capacity of 6 cubic feet of mixed concrete. The materials were charged into the mixer in the following general order, subject to minor variations introduced by the different men in charge of the mixing: (1) 2 cubic feet coarse aggregate, (2) 2 or 3 cubic feet of sand, (3) 1 bag Portland cement, (4) 2 or 3 cubic feet of coarse aggregate, (5) water. Before admitting the water the materials were mixed dry for a period varying from ¼ to ½ minute, the total time of mixing being limited to 1½ minutes as a maximum and 1 minute as minimum. The water was measured by means of a gage glass and scale, the former connecting with a vertical measuring tank placed above the mixer (Fig. 12).

The concrete was discharged into wheelbarrows which were raised to the platform of the staging for discharge into the forms. The concrete was spaded along the inside of the form and the latter was tapped with a hand hammer to assist in obtaining a good concrete surface.

To obtain workmanship comparable with that on field-placed concrete, several experienced men connected with local construction companies were at different times placed in charge of the mixing and placing, about one-half of the total number of concrete coverings and columns being thus placed. The methods thus introduced were followed in the mixing and placing of the concrete for the remaining columns.

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2. PLASTER ON METAL-LATH PROTECTIONS

(*a*) PLACING OF LATH

For test No. 23 the lath for both the outer and the inner layer was supported on round bars held upright by iron clips made from ⅛ by 1 inch flat bars and secured to the structural-steel section. This method proved very cumbersome, and the lath for the other steel columns was supported on ⅜ by ¾ inch channels held in vertical position by wire ties. The high-ribbed lath was supported directly on the metal.

The lath was placed around the column in horizontal courses with laps of about 3 inches. Horizontal joints between sheets had generally shorter laps. All laps were wired with No. 18 wire ties placed 3 to 6 inches apart vertically and one on the middle of each side of the horizontal joints.

(*b*) APPLYING THE PLASTER

The plaster coats were of the maximum thickness practicable with the given mixture of plaster. The first coat of a layer was allowed to set two or three days before applying the second coat.

The lathing and plastering were done by experienced men obtained through a local plastering contractor.

3. HOLLOW CLAY-TILE AND BRICK PROTECTIONS

(*a*) PROPORTIONING OF MORTAR

The mortar was proportioned by volume parts of loose materials. The proportion used was 1 part Portland cement, 1 part stiff lime putty (slaked lump lime), and 4 parts fine bank or lake sand.

(*b*) PLACING OF TILE AND BRICK

The coverings were detailed in advance, and tile of the required size was supplied when possible. All cutting of tile and brick was done on the job with the hammer or trowel. A view of tile and gypsum-block protections under construction is shown in Fig. 13.

The work was done on a contract basis by a mason contractor, and it is thought that the workmanship obtained approximates that secured in good building practice.

(*a*) PLACING OF CONCRETE FILLING

Where concrete filling between the tile and column was used, the tile was held in place by clamping both ways every 2 feet, boards being placed along the protection inside of the clamps. The filling was placed from the platform of the staging shown in

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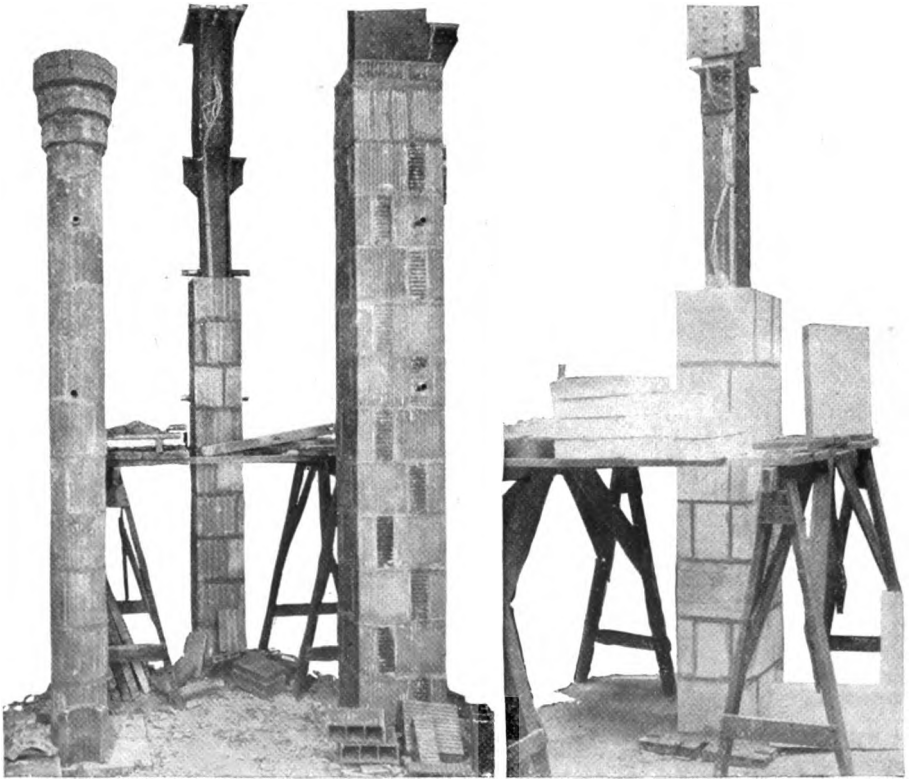

FIG. 13.—*Placing of clay-tile and gypsum-block protections*

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Fig. 11, the whole space being filled in one continuous operation. The clamps effectively held the tile against the pressure of the wet fill, and very few mortar joints were broken from this cause.

4. GYPSUM-BLOCK PROTECTIONS

(a) PROPORTIONING OF MORTAR

The proportion of mortar used, 1 : 3, neat, unfibered, calcined gypsum and fine lake sand, was the richest mixture that would work satisfactorily under the trowel, the materials being measured by loose volume.

(b) PLACING OF BLOCK

The blocks were cut from standard-size partition blocks with a hand saw, the resulting pieces being generally all used either in the covering or filling. The mortar joints in the covering and between the blocks and flanges of the test column were well filled.

(c) PLACING OF FILLING

Where wet filling was used the dry materials were first turned three times by hand and then mixed with water in small batches. The filling was placed as the blocks were laid up, two courses being generally filled at one time.

The placing and filling of the gypsum coverings were done by a fireproofing contractor employing men experienced in handling the given material.

5. METHOD OF STORAGE

Normal air storage was used for all columns and for the greater number of auxiliary test specimens.

The test columns were stored in the testing room, the temperature of which during the summer months was about the same as that of the outside air, and varied during the winter months from 5 to 25° C.

The auxiliary test specimens, consisting of 8 by 16 inch concrete cylinders and 2-inch mortar cubes made from the material of the coverings as placed, were arranged in tiers and stored near the test columns. In some laboratory tests with mortar, the briquets and cubes were stored for periods in damp closet or water as stated in Section V (Table 28, Appendix D, p. 356).

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V. AUXILIARY TESTS OF MATERIALS

The results of the auxiliary tests give information on the physical, chemical, and thermal properties of the materials used in the columns and coverings. Where generally accepted standards in the form of specifications exist some comparable measure of quality is thereby afforded. For the majority of the materials no general specifications have as yet been developed, and their representative character must be determined by the extent of their use and the methods employed to obtain material of average quality.

The large number of tests of concrete, mortar, and plaster give important information on their properties and variability as made under conditions approximating those obtaining in building construction.

1. TESTS OF STRUCTURAL, BAR, AND WIRE STEEL

Results of tension tests are given in Tables 5 and 6 (Appendix D).[1] The specimens of structural steel, about 7 inches long and ⅜ to ½ inch wide, were cut before test from the upper, enlarged portion of the column section by drilling and sawing. They were finished to uniform width over a gage length of 2 inches. Tests of bar steel were made on the full sections of the bars used except as noted in Table 5 (p. 336).

Specimens for hardness tests were taken where tension specimens were difficult to obtain, and results of tests are given in Table 7 (p. 338).

Chemical analyses of structural and rivet steel are given in Table 8 (p. 339).

The column steel was of the grade generally used for structural purposes. Of the 77 structural-steel columns used in the tests, 36 were donated by the manufacturers and 22 were bought from the same sources, with no specifications as to quality of material. The steel for the remaining columns, which were purchased at a later date, was specified to conform with the specifications for structural steel for buildings of the American Society for Testing Materials. Correspondence with the manufacturers indicates that all of the steel was made by the open-hearth process and that,

[1] The tabular matter for this section is placed in Appendix D (p. 334-364).

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with the exception above noted, the specifications followed were manufacturer's standard specifications for steel for railway bridges or for medium open-hearth steel, tensile strength 55 000 to 70 000 pounds per square inch. A few of the test results are above or below the specification limits by 10 per cent or less, a variation that may be allowable, considering that most of the specimens secured were smaller than the standard size.

The metal for the reinforcing bars was specified to conform with specifications for billet-steel concrete reinforcing bars of the American Society for Testing Materials, structural grade. The spiral hooping was of hard-drawn wire of high tensile strength (87 400 pounds per square inch).

2. TESTS OF CAST IRON

Results of transverse and tension tests are given in Table 9. The specimens of the horizontally cast columns were cut from the projecting flanges in the upper 3 feet or head of the column. Those representative of the metal in the vertically cast-iron pipe columns 10A, 114, and 115 were cut from a duplicate column. Results of chemical analyses of the iron in this column are given in Table 10 (p. 339).

The iron for the horizontally cast columns was of gray foundry pig with admixture of machinery casting scrap. The mixture used for the cast-iron pipe column and caps was the same as that regularly used in the manufacture of cast-iron water pipe. The following analyses of the mix were furnished by the makers:

	Horizontally cast columns	Vertically cast—			Horizontally cast columns	Vertically cast—	
		Columns	Caps			Columns	Caps
Silicon	2.04	1.60	1.70	Phosphorus	.714	.....	.....
Manganese	.45	.34	.32	Sulphur	.108	.081	.069
Combined carbon	.72	.....	.....				

For the iron in the horizontally cast columns, the analyses and test results indicate conformity with accepted specifications for gray-iron castings of medium weight. For the vertically cast columns, tests of specimens cut from one end of the duplicate column gave transverse and tension values about equal to the specification limits and those of specimens cut from the other end gave values lower by about 15 per cent.

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3. TESTS OF PORTLAND CEMENT

Tests of the Portland cement used in the columns and coverings were made by the Washington and Pittsburgh laboratories of the Bureau of Standards and by the R. W. Hunt Co. The results are given in Tables 11 to 13 (Appendix D). Sample Nos. 1 to 5 and B–1 and B–2, Table 11; H–1 to H–6, Table 12; and H–1 to H–3, Table 13, were all from the mill shipment received in April, 1916. Sample Nos. H–7 and H–8, Table 12, were of a later purchase of the same brand as the original mill shipment. Sample No. 12, Table 11, was of the Portland cement used in filling the pipe column of test No. 12. All samples were individual sack samples taken from the portions of the shipment used at the given time.

The time of setting was determined with the Gillmore needles in all tests. Other details of the test conformed with the specifications published by the American Society for Testing Materials, 1916. As based on average results, the tests indicate conformity with the given specifications within the pertaining limits and tolerances, excepting the test of sample No. 12. The latter gave low results in points of fineness and seven-day tensile strength.

4. TESTS OF SAND

The chemical and physical properties of the concrete and finer sands are given in Tables 14 to 17 (pp. 342–344).

The specific gravity was determined by measuring the volume of benzene displaced by a given dry weight of sand in a Le Chatelier apparatus.

The weight per cubic foot is that of the loose dry sand poured into the measure without shaking or bumping and leveled off even with its top.

The weight of the dry contents of a cubic foot of the sand as used on the work was somewhat less than that given in the tables due to the moisture contained which caused it to assume greater bulk than in the dry condition, the difference being about 10 per cent with coarse sand of 3 per cent moisture content. With the finer sands the decrease in weight was about 25 per cent as caused by a moisture content of 4 per cent.

The percentage of computed voids was taken equal to the following relation of values:

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$$100\left(\frac{\text{Weight per cubic foot (dry)}}{1-\text{specific grav.}\times\text{weight of cubic foot of water}}\right).$$

The granular analysis gives percentages, by weight, passing the given sieve openings.

5. TESTS OF COARSE CONCRETE AGGREGATES

Chemical analyses and physical properties of the broken stone, gravel, and cinder used for concrete aggregate are given in Tables 18 and 19 (p. 344).

The physical properties were determined according to the methods given for tests of sand, except that apparent specific gravity was determined in water in a graduated vessel on the saturated aggregate and was taken as the ratio of the dry weight to that of the water displaced.

6. SOURCE AND CLASSIFICATION OF CONCRETE AGGREGATES

The sands and coarse aggregates used for concrete were analyzed for mineral composition and their chief constituents are given in Table 20 (p. 345). Further information on their mineral composition, geological origin, geographic location, and the method of preparing them for commercial use are given in the following descriptions:

(*a*) FOX RIVER SAND

This sand was obtained from a sand and gravel deposit about 3 miles east of Elgin, Ill. After crushing of the oversized gravel the whole is washed and screened to size. The grade is known in the Chicago market as coarse "torpedo" sand, the general maximum size of grains being ¼ of an inch. It is clean and very sharp, only 15 per cent of the grains being rounded.

The sand is of glacial origin, and its principal mineral constituents are calcite and dolomite, 44 per cent; quartz and chert, 39 per cent. Ferromagnesians, chiefly biotite, pyroxene, and hornblende, constitute 4 per cent. No free clay matter was present, although about 2 per cent of hydrous aluminum silicates were contained in the calcite and dolomite.

(*b*) JOLIET SAND

This is a clean, sharp, glacial sand obtained from a sand and gravel deposit at Rockdale, Ill., the method of preparation and

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grading being the same as for Fox River sand. The chief mineral constituents are calcite and dolomite, 47 per cent; quartz, 42 per cent; feldspar, chiefly orthoclase ($KAlSi_3O_8$), 7 per cent; ferromagnesians, chiefly as biotite, mica, and hornblende, 3 per cent.

(c) MERAMEC RIVER SAND

This sand is obtained incidentally to screening and washing of gravel dredged from the bed or banks of the Meramec River at Drake, Mo. It consists almost wholly of quartz and chert grains formed from sandy chert by rolling in the stream bed. A small amount of calcite (1 per cent) is present as a coating on some of the grains. The quartz grains are fully rounded, while the chert is partly rounded or subangular.

(d) LONG ISLAND SAND

This sand was obtained from banks near Roslyn, Long Island, N. Y., and was screened and washed. It is a glacial sand formed from the crystalline rocks of New England. The principal constituents are quartz and feldspar, with minor amounts of ferromagnesians, slate, magnetite, and cinders. The cinders are not an original constituent of the sand. The sand is clean, sharp, and angular, the grains showing little rounding.

(e) PELEE ISLAND SAND

This sand was dredged by boat from the bottom of Lake Erie, at Fish Point, Pelee Island, Ontario. It is of glacial origin with sharp and angular grains that show almost no rounding. Of the mineral constituents, calcite and dolomite constitute 32 per cent; chert and quartz, 51 per cent. Clay is present to the extent of 4 per cent, of which about one-half is free clay matter and the rest shale.

(f) PLUM ISLAND SAND

This sand was dug on a beach at Plum Island on the Massachusetts coast and was not screened or washed. It is of glacial origin although much modified by ocean waves and currents. About 50 per cent of the grains have been worn to a subangular condition and about 3 per cent are well rounded. It is fine-grained and contains no free clay material, the mineral composition being otherwise closely identical with that of the Long Island sand.

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(g) CHICAGO LIMESTONE

The stone, which was crushed to nominal ¾-inch size was quarried at Gary, Ind., from a ledge of the Niagara formation. It is a true dolomite (calcium magnesium carbonate), carrying a moderate amount of clayey impurities.

(h) JOLIET GRAVEL

This is obtained from the same source as Joliet sand. It is a glacial gravel, consisting largely of limestone pebbles, of which 20 per cent are rounded and 60 per cent subangular or partly rounded. Mineralogically the composition is 86 per cent dolomite, containing about 5 per cent clayey impurities; 5 per cent quartz in the form of sandstone and quartzite; 2 per cent quartz, 5 per cent feldspar, and 2 per cent ferromagnesian silicates as pebbles of granite, gabbro, and other basic igneous rocks.

(i) MERAMEC RIVER GRAVEL

The pebbles consist almost entirely of chert, an amorphous form of silica containing a variable amount of water in chemical combination. In the sample examined, 85 per cent of the pebbles show no trace of an earlier structure, 4 per cent are sandstone replacements, 9 per cent are limestone replacements, and 2 per cent have once been shale. The pebbles are colored brown with limonite ($2Fe_2O_3 \cdot 3H_2O$).

(j) NEW YORK TRAP ROCK

This was quarried at Haverstraw, N. Y., from the northern end of the sill of diabase which extends along the western shore of the Hudson River from this point to Staten Island, forming the Palisades of the Hudson. It is an intrusive igneous rock of finely crystalline texture. Of the minerals present, feldspar, chiefly plagioclase ($CaAl_2Si_2O_8$), with some albite feldspar ($NaAlSi_3O_8$), constitute 71 per cent; ferromagnesians as augite, 19 per cent, and as olivine, 7 per cent. In addition to 2 per cent of magnetite (Fe_3O_4), there are present scattered crystals of quartz (SiO_2) and apatite [$Ca_5F(PO_4)_3$].

(k) ROCKPORT GRANITE

This stone was quarried at Rockport, Mass., and is sometimes known as Cape Ann granite. It is an intrusive igneous rock that crystallized under great pressure from a fused condition.

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The mineral composition is 59 per cent feldspar, chiefly orthoclase ($KAlSi_3O_8$) with some albite ($NaAlSi_3O_8$); 35 per cent quartz; 5 per cent ferromagnesian silicate, as hornblende [(Fe, Mg)SiO_3].

The feldspars show slight alteration to kaolin and chlorite. The quartz contains many tiny cavaties filled with gases and water occluded when the rock was formed.

(*l*) CLEVELAND SANDSTONE

The stone was quarried at Amherst, Ohio, and geologically is known as Berea sandstone or Berea grit. It is pure sandstone, 98 per cent consisting of subangular grains of quartz.

(*m*) HARD-COAL CINDERS

This is an anthracite cinder representative of the product used for cinder concrete in New York, N. Y. It consists largely of a porous, fused mass of basic silicates, apparently high in lime and magnesia and low in iron. There is present about 10 per cent of unburned coal and 5 per cent unfused ash.

7. TESTS OF CONCRETE

The quality of the concrete secured is thought to represent that of field-placed concrete as nearly as it can be conveniently duplicated under laboratory conditions. The method used for proportioning the dry materials was more accurate and the time of mixing longer than now generally obtain on construction work. The range of consistency resulting from the method employed for mixing the concrete is deemed to be representative of current field conditions. Where concrete is properly placed in large building members its properties are probably more uniform than as given by tests of the relatively smaller cylinders, being for a given mixture and consistency, more nearly the average values obtained in the tests.

(*a*) TEST SPECIMENS

Four cylinders of 8-inch diameter and 16-inch length were molded of the concrete mixed for each concrete covering, filling, or column. Cylinders were also taken of the concrete in the head protections of some of the columns where the method of mixing had been modified so as to give information on the effect of a lower water content and of a longer mixing period.

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The molds were of cast iron with machined base plates. After the concrete had set and before removal of the molds, the cylinders were capped with a plastic mixture of Portland cement and calcined gypsum applied even with the top of the molds by means of a finished iron plate which remained in place until the mixture had set. In molding, care was taken to obtain the same consistency of concrete in the cylinders as in the column. Except for spading with a thin tool near the surface, the concrete was not puddled or tamped in the mold.

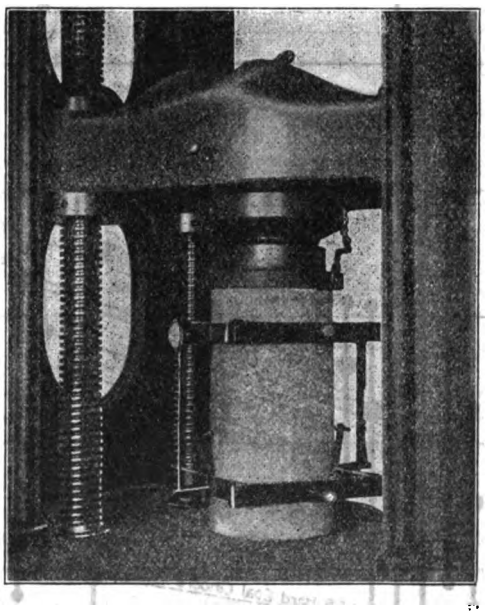

FIG. 14.—*Concrete cylinder after test*

(*b*) PER CENT WATER IN CONCRETE MIXTURE

The water introduced in mixing was measured at the mixer, and, for all but the first 15 columns covered, that present in the aggregates was determined from samples taken on the day the concrete was mixed. The dry weight in pounds per cubic foot of the aggregates as used was determined in a number of separate tests with each aggregate in which the same methods of filling the measures were used as were employed in proportioning materials for the concrete. The net weight per bag of Portland cement was found to be very nearly 94 pounds. The water content of the

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mixture is the sum of the water introduced, and that contained in the aggregates and in the tables and diagrams is expressed as a percentage of the total dry weight.

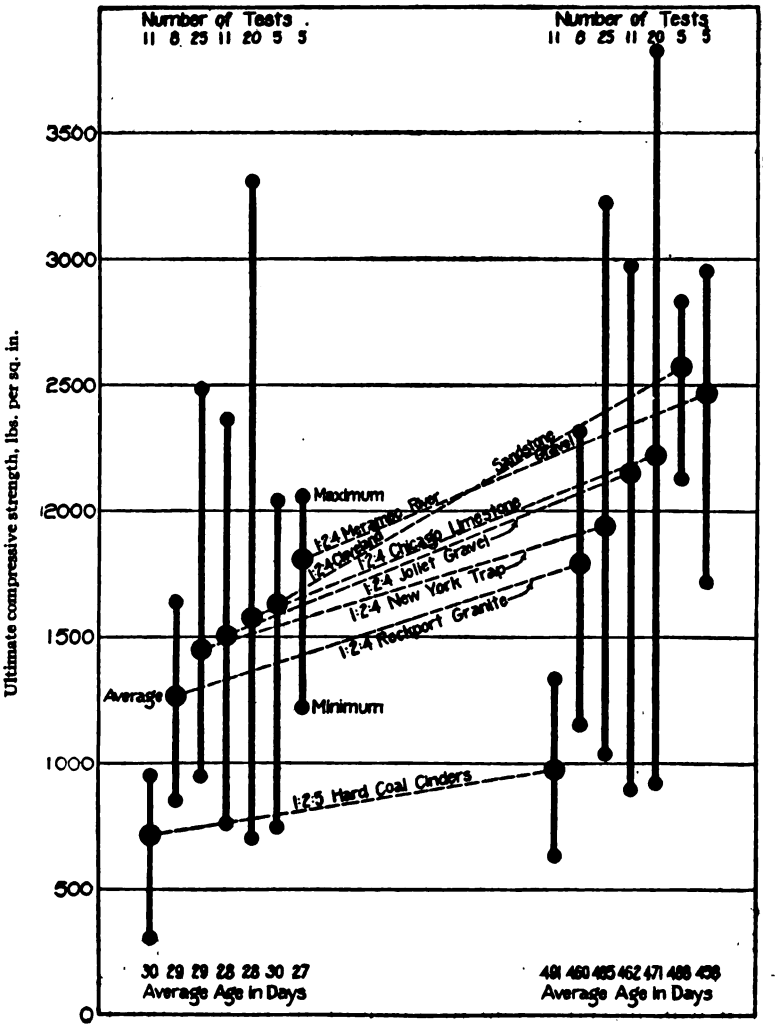

FIG. 15.—*Average and range of compressive strength of 1:2:4 and 1:2:5 concrete*

While the above method gives a fair comparison in point of consistency of concrete made from the same aggregate, the comparison between those of different aggregates is less direct, due to differences in specific gravity and absorption.

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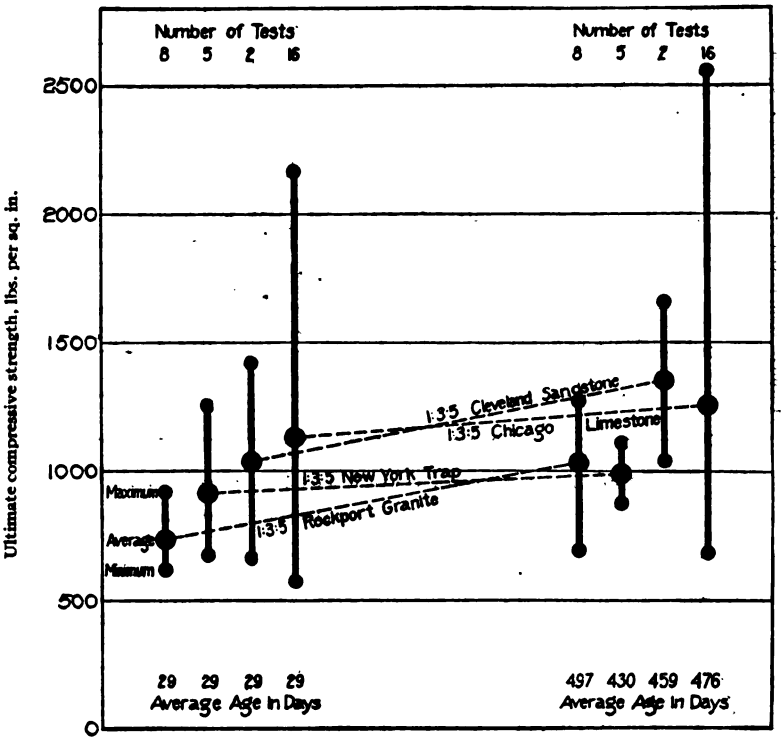

FIG. 16.—*Average and range of compressive strength of 1:3:5 concrete*

(*c*) **TESTING OF CONCRETE CYLINDERS**

Two cylinders of each set were tested in compression at the age of four weeks and two at the time the corresponding column was tested. In the case of the latter, the compressive load was applied by increments of about 10 000 pounds, and the deformation measured over a 10-inch gage length by means of an Ames dial gage mounted in a pivoted frame (Fig. 14). The results of tests on concrete mixed for the protections and columns are given in Table 21 (p. 346) and those of the concrete in some of the head protections in Table 22 (p. 350).

(*d*) **COMPRESSIVE STRENGTH OF CONCRETE**

The diagrams in Figs. 15 and 16 give the average and range of compressive strength of the cylinders taken from the concrete mixed for the columns, their covering and filling. In Figs. 17 and 18 the general effect of consistency on compressive strength is

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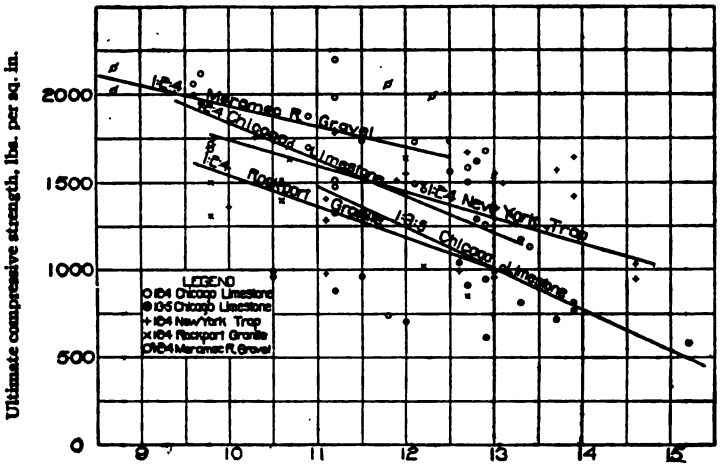

FIG. 17.—*Effect of consistency on compressive strength of concrete. Average age, 28 days*

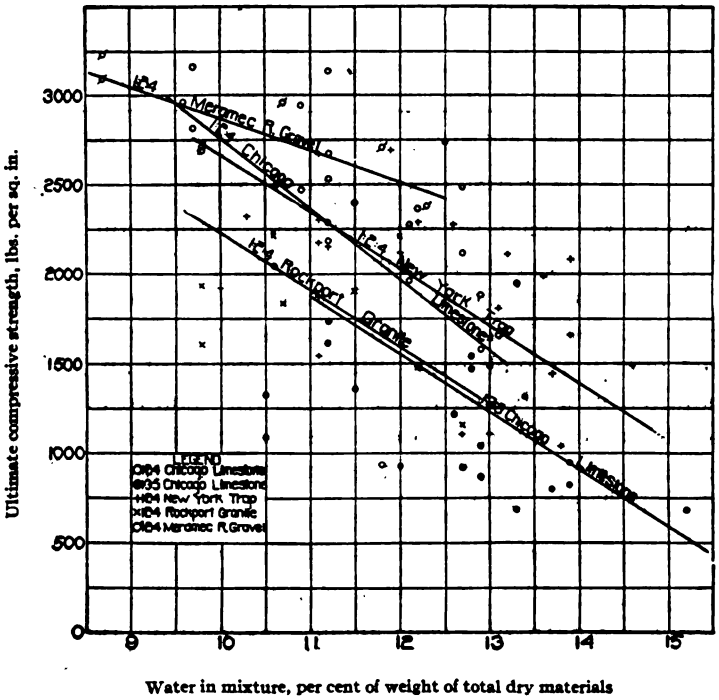

FIG. 18.—*Effect of consistency on compressive strength of concrete. Average age, 490 days*

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shown. Herein are also included the tests (Table 22) made to study the effect of a lower water content. Fig. 19 gives a comparison of the strength of concrete mixed for 1 and for 2 minutes.

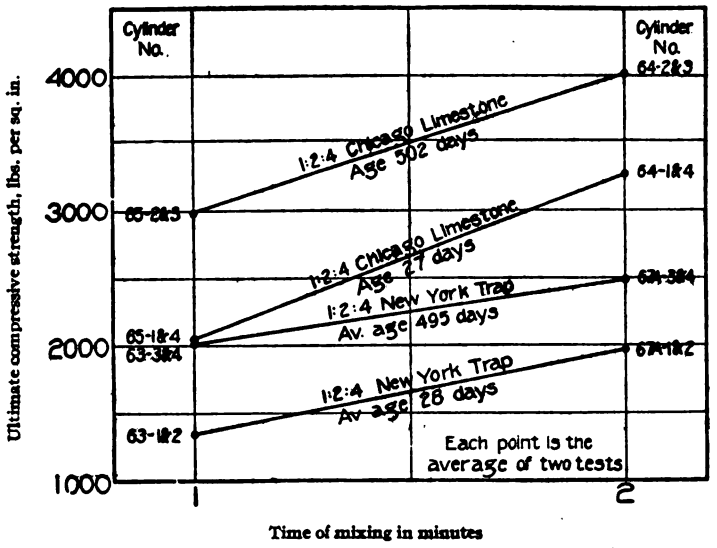

FIG. 19.—*Effect of time of mixing on compressive strength of concrete*

The mixtures having a water content of 9 to 11 per cent were of quaking or mushy consistency, the drier ones being discharged from the mixer with considerable difficulty. The higher water percentages (12 to 15) represent varying degrees of fluid consistency. The restricted space in which some of the concrete was placed made fluid or semifluid consistencies necessary in order to secure proper placement, although beyond the point where water collects above the concrete after discharge little is gained in this particular by the excessive use of water. For a given consistency the cinder and, to less extent, the sandstone, required a larger amount of water than the other aggregates. due to greater absorption.

For the concrete in the columns and coverings (Table 21) the amount of water introduced was determined by the man in charge of the mixing and placing of concrete for the given column, the water for the first batch being added by increments until the desired consistency was attained. For the concrete in the head

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protections (Table 22) the amount of mixing water necessary for any desired total moisture content was generally predetermined, knowing the dry weight of the aggregates employed and their moisture content at the given time.

(e) MODULUS OF ELASTICITY OF CONCRETE

A comparison of values for 450, 650, and 850 pounds per square inch is given in Fig. 20 as obtained from Table 21. The modulus of elasticity is taken as the ratio of the given unit stress to the corresponding total unit deformation. In Fig. 21 the variation with ultimate compressive strength is shown, and in Fig. 22 the effect of water content of the mixture. The plots are based on values of modulus of elasticity given in Tables 21 and 22 for stress of 650 pounds per square inch.

In all tests the age of the concrete was over one year and the values of modulus of elasticity found are on the average higher than those generally taken to obtain for concrete at lower age.

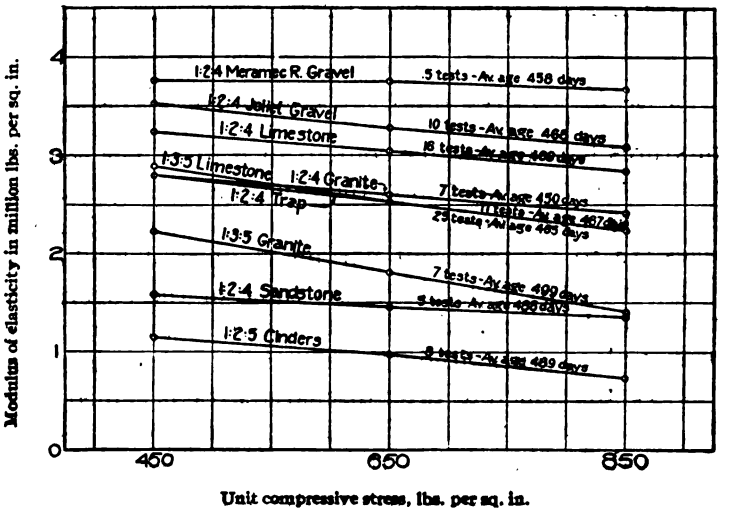

FIG. 20.—*Modulus of elasticity of concrete at 450 to 850 lbs. per sq. in.*

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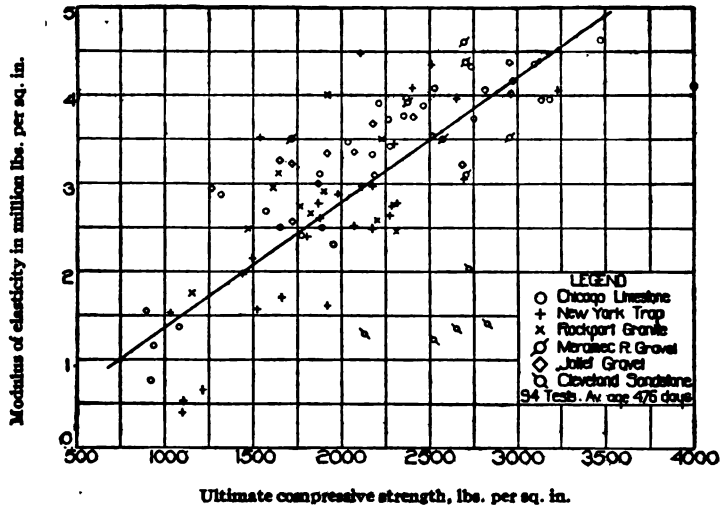

FIG. 21.—*Modulus of elasticity of 1:2:4 concrete at 650 lbs. per sq. in. Variation with ultimate compressive strength*

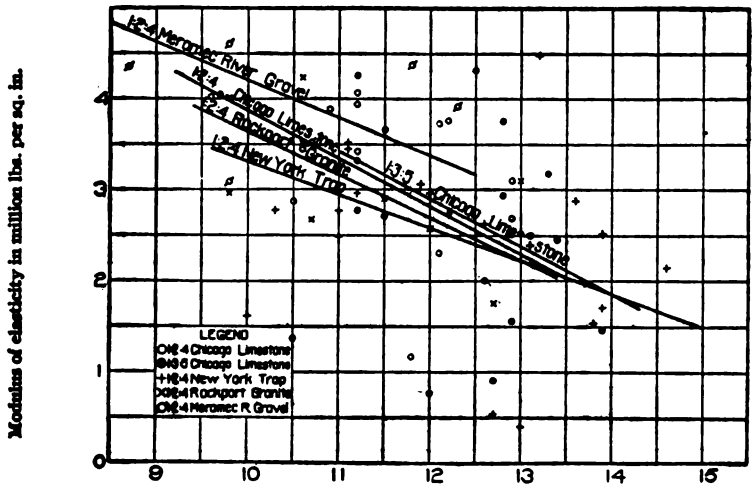

FIG. 22.—*Effect of consistency on modulus of elasticity of concrete at 650 lbs. per sq. in.*

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8. TESTS OF LIME

Chemical analyses and physical properties of quicklime and hydrated lime are given in Table 23. The analyses disclosed a high calcium lime. The quicklime passed standard specifications but the hydrated product failed in point of CO^2 content, fineness, and soundness. The latter was used only as a minor component in the Portland-cement plaster protections.

9. TESTS OF CALCINED GYPSUM

Analyses and tests of eastern and western gypsum are given in Table 24 (p. 351).

10. TESTS OF MORTAR, PLASTER, AND FILLING

Results of compression tests on the mortar and plaster used in the coverings are given in Tables 25 to 27 and a further comparison of average and range of compressive strengths is given in Fig. 23 The specimens for these tests were taken of the materials as mixed by the workmen. They were stored in air without artificial drying or curing.

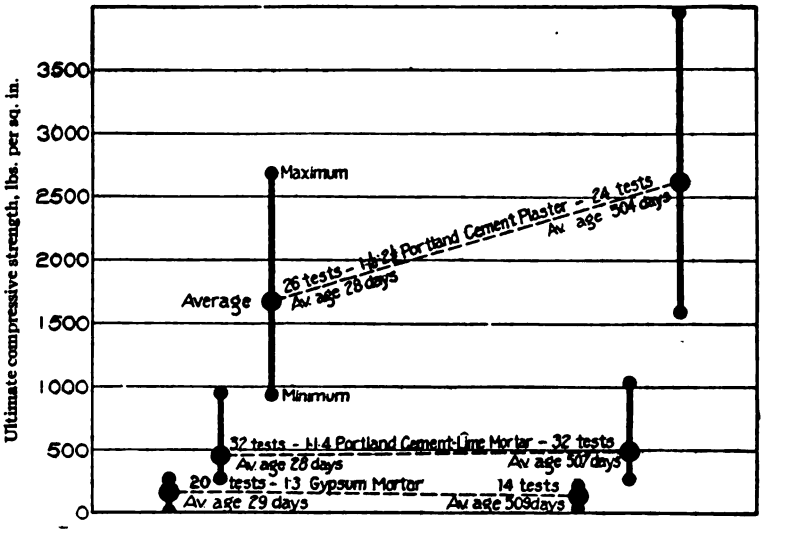

FIG. 23.—*Average and range of compressive strength of mortar and plaster*

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The results of a series of mortar tests made by the Pittsburgh laboratory of the Bureau of Standards are given in Table 28 (p. 356). The same materials, proportions, and average percentage of water were used as for the mortar and plaster of the column coverings.

Table 29 (p. 354) gives results of compression tests on cylinders molded of the mixture with which gypsum-block protections were filled.

11. TESTS OF HOLLOW CLAY TILE

(*a*) CLASSIFICATION AND DESCRIPTION

The characteristics of the hollow tile employed as column covering are given in Table 30 (p. 357).

The straight partition tiles, Nos. A to E, were nonporous; that is, burnt without sawdust or other filling. The curved tiles, Nos. F, G, and H, in point of method of manufacture, were porous.

(*b*) POROSITY AND ABSORPTION

The porosity of the burnt clay (Table 31) was obtained by drying samples of tile to constant weight at 112° C and immersing in hot water under vacuum of 24 inches of mercury for about four hours, the porosity being based on the ratio of volume of water absorbed to that of the test specimen. The weight of water absorbed is also expressed in Table 31 (p. 358) as a percentage of the dry weight.

(*c*) COMPRESSIVE AND TRANSVERSE STRENGTH

Table 31 gives results of compression tests of all types of hollow tile used in the column coverings. The curved tile was tested on end and the partition tile was tested both on end and on edge. By the latter method the load was sustained by the outer shells only and the average maximum unit loads developed are in all cases lower than for the tests made with the tile on end.

The transverse tests (Table 32) were made with center load and supports 10 inches apart. Failure generally occurred on inclined planes extending from points on the lower surface upward toward the loading line. In calculating the maximum outer fiber stress and horizontal shear, the moment of inertia and other properties of the section were obtained, using the measured dimensions of each test specimen.

(*d*) TEMPERATURES OF VITRIFICATION AND FUSION

In determining temperatures of vitrification a number of pieces of each kind of tile were dried and weighed as for the porosity

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tests, and heated to 800° C in a muffle furnace, after which samples were withdrawn at temperature intervals of 30° C, allowed to cool slowly, and tested for porosity. The temperature of vitrification was taken as the point or region of minimum porosity.

In preparing samples for the fusion test, fragments of a number of tiles from the same clay were broken up and a sample obtained by quartering. This was ground and again sampled, the final sample being ground to pass through an 80-mesh sieve and made into cones ⅞ of an inch high. These were fired in a small pot furnace between standard cones, the temperature of the furnace being determined with platinum thermocouples. Softening was indicated when the tip of the test cone went over. Fusion was taken as occurring at the stage when the tip of the cone reached the level of its base.

The temperatures of vitrification and fusion determined for the hollow tile and brick are given in Table 33 (p. 361).

12. TESTS OF BRICK

Results of tests giving the porosity, absorption, compressive and transverse strength of the brick used in the brick protections are given in Tables 34 to 36 (p. 361–362).

The brick was made in the Chicago district of calcareous surface clay and burnt to medium hardness, the color being generally light red. The average dimensions in inches were 2¼ by 3¾ by 8.

13. TESTS OF GYPSUM BLOCK

Results of porosity, compression, and transverse tests of gypsum block are given in Tables 37 to 39 (p. 363–364).

The porosity tests were made on pieces of block dried for 24 hours at 60° C and saturated in kerosene under vacuum, the method being in other respects the same as given above for clay tile.

In calculating modulus of rupture the outside dimensions of the block were used, the scoring and imprints being neglected.

14. TESTS OF GYPSUM WALL BOARD

Results of transverse tests on gypsum wall board, in the dry condition and after saturation in water, are given in Table 40. The strength of the board was limited by the tensile strength of the paper facing, the latter developing greater resistance when the strain was parallel with the grain of the paper.

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VI. DESCRIPTION OF FURNACE AND RELATED EQUIPMENT

The furnace and accessory equipment used in the tests were specially designed for the purpose and consist of a carriage and traveling crane for handling the test columns, a furnace in which the columns are subjected to fire, a ram with restraining frame for applying load, and hydrant with means for appplying water in the fire and water tests.

1. BUILDING

The tests were made at Underwriters' Laboratories, Chicago, Ill., in a building designed for work of this character, a partial plan of which is given in Fig. 24 and a sectional elevation through the center in Fig. 25.

The central portion of the building, wherein the furnace for testing columns is located, is in one story about 37 feet high, an intermediate operating floor being located in the outer bays. Sliding skylights in the roof of the building above the furnace provide means for ventilation. The operator's station on the upper floor is inclosed within fire-resistive partitions, with openings protected by wired glass.

2. APPARATUS FOR HANDLING COLUMNS

A carriage traveling on the lower flange of the I beams placed above the storage bays for the columns is used for transferring the latter to the traveling crane. The column to be moved is lifted on threaded rods passing through the top plate of the column and plates supported on a truck contained within the carriage. This truck carries the column from the traveling crane to its location in the furnace, on I beams (*a*, Fig. 25) spanning the space above the furnace (Fig. 24).

The column is finally attached to the head of the loading ram and lowered into position.

3. LOADING APPARATUS

(*a*) LOADING RAM

The special hydropneumatic ram used for loading the columns was designed to maintain a constant load during the test and to develop characteristic deformation at the point of failure.

The main ram is 36 inches in diameter and is connected with a smaller lifting ram supported on its upper head. The operating

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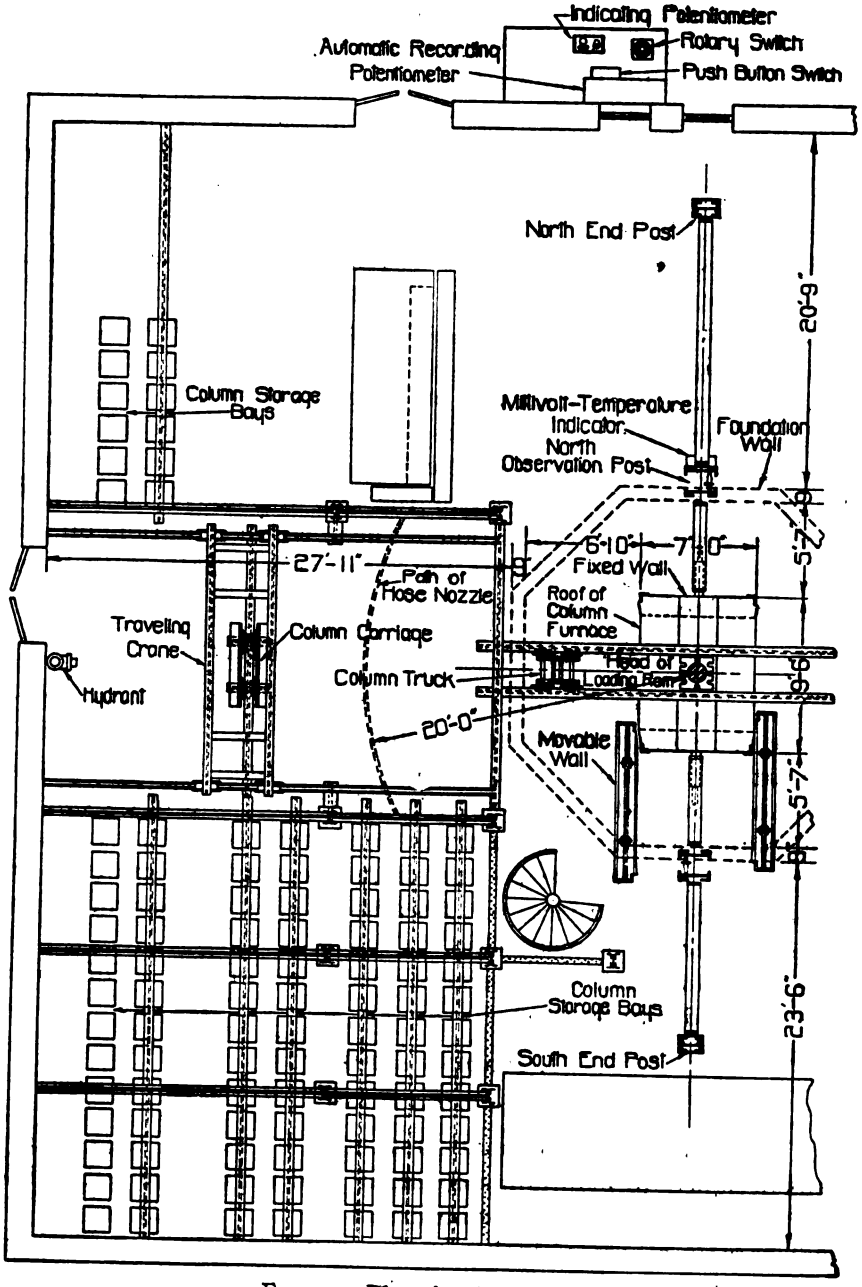

FIG. 24.—*Plan of testing room*

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medium is water maintained at the required pressure by an air compressor which discharges into steel pressure tanks connected with the ram cylinder through a 6-inch main. A pump is also provided to maintain the water at any given level.

Provision is also made for applying load by pumping water directly into the main cylinder. The gate valve in the main leading to the air-pressure tanks is in this case closed and regulation of pressure obtained with a spring relief valve.

(*b*) CONTROLLING DEVICES

A gate valve is placed in the pipe connection between the pressure tanks and the main cylinder, and also an automatic cut-off valve which shuts off the pressure tanks and releases the pressure in the main cylinder after a predetermined downward movement of the plunger of the ram has taken place. For the present tests the valve was set to begin shutting off pressure after the plunger had gone down 2 inches, the valve releasing the pressure after an additional 1¼-inch downward movement. This caused maximum center deflection of the column at failure of about 15 inches. As a further precaution a pressure of not less than 400 pounds per square inch was maintained in the lifting cylinder.

The portion of the main cylinder chamber below the piston is connected with the outside air and also with a closed discharge chamber, with the latter through perforations in the cylinder lining. A depth of water is maintained in this part of the cylinder to act as a cushion in case the travel of the ram piston, following a sudden column failure, should extend to the end of the cylinder.

To obtain an approximate record of the load sustained by the column during test and during the period of failure, a steam-engine indicator was mounted on the control board (*s*, Fig. 27) with its pressure chamber connected with the main cylinder of the ram and the drum actuated by a cord attached to the ram plunger. The resulting card diagram indicated on axes at right angles, the pressures in the main cylinder and the corresponding positions of the ram plunger.

For convenience in operating, the valves for admitting and releasing pressure to all parts of the loading system are located at the control board (Fig. 27). Here are also located the load and pressure indicating gages and the automatic and manually oper-

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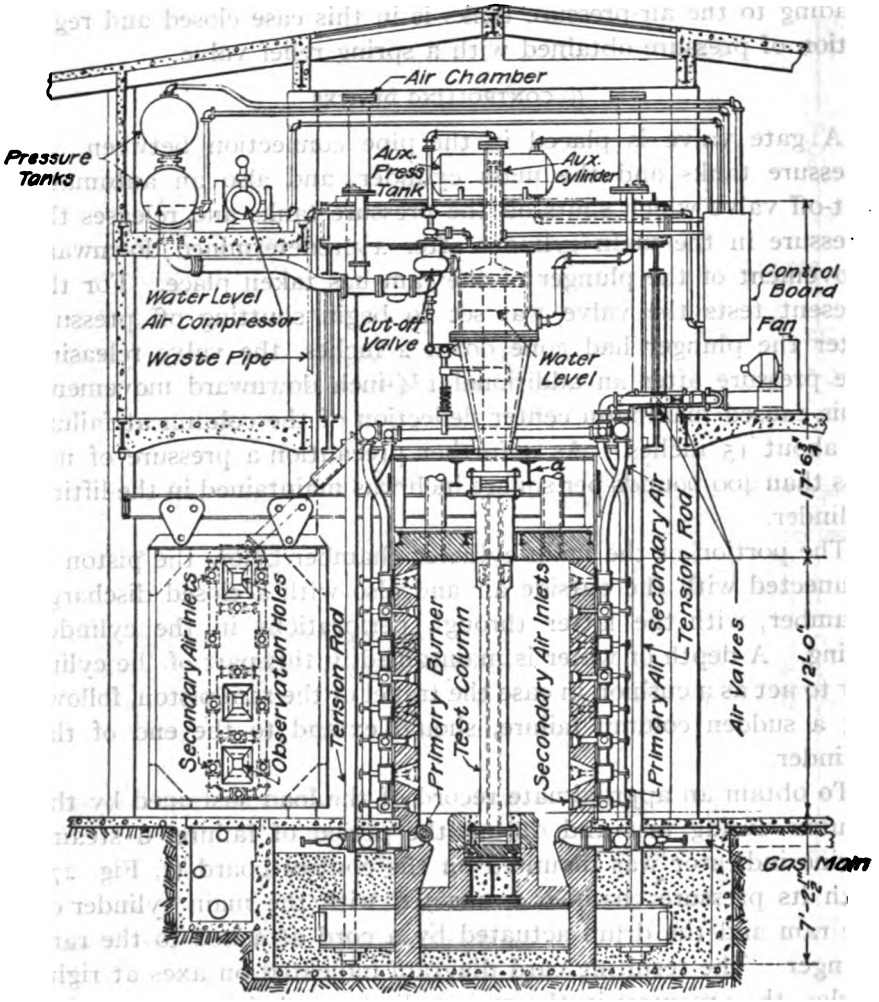

FIG. 25.—*Elevation of testing machine*

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ated controls for the starting switches of the air compressor and water pump. The main features of the installation are shown in Fig. 25, minor piping and details being omitted.

(c) RESTRAINING FRAME

The ram is supported from the upper transverse members of the frame, the lower set being embedded in the foundation pit. Each set consists of two pairs of 24-inch I beams with cover plates. Two vertical tension bars on each side carry the reactions from the applied column loads. The lower end of the ram is supported laterally in an intermediate cross frame.

(d) BEARING DETAILS

The ends of the test column are bolted to rolled-steel plates 2 inches thick, which are in turn bolted to a cast-steel foundation plate at the bottom and to the lower flanged head of the ram at the top. Between these and the column plates are placed adjustable bearings consisting of skew disks, the middle two of which can be turned to obtain even bearing.

(e) CAPACITY AND CALIBRATION

The machine has a load capacity of 545 000 pounds and has been calibrated by the Bureau of Standards, using a comparison bar whose load-deformation relation was determined in the 2 300 000 pound Emery testing machine at Washington.

The accuracy of loading within the range used in this series of tests can generally be taken to be within 1 per cent of the applied load, with a possible extreme variation of 2 per cent. The errors incurred are due to variation in the ram friction and in the indication of the test gages used for determining the load. The latter were calibrated in a dead-weight gage tester before or after each test.

4. TESTING FURNACE

(a) COMBUSTION CHAMBER

The chamber is 7 feet square and 12 feet high, exclusive of two shallow pits to receive falling material and carry off water during the fire-stream tests. The sides are formed by two stationary brick walls and two movable brick walls suspended in structural-steel frames from overhead beams by trolleys (Fig. 25). These walls are pulled open by means of a motor-driven hoist. The top of the furnace consists of heavy fire-clay blocks supported

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by steel I beams and hangers. Some of the blocks are removable to permit installation of the test column. Concrete made of Portland cement and crushed fire brick and reinforced with rods and wire mesh was later substituted for fire clay in making additional roof blocks. The bottom of the chamber is formed by the protection provided for the lower bearing plate and the supporting members of the restraining frame.

The products of combustion are carried off through four flues extending from the furnace through the roof of the building. The furnace walls are provided with mica-glazed observation holes, so arranged near the top, middle, and bottom that all parts of the test column can be observed.

A general view of the furnace with a preliminary test column in place is given in Fig. 26.

(*b*) BURNERS

The furnace is heated by means of four primary blast burners extending in at the corners at the bottom of the chamber, and are all arranged to discharge in an inclined direction upward and toward the adjacent corner. The burners are supplied with gas and air through tubes connecting with mains located in a shallow pit under the floor outside of the furnace. Gas is supplied to the furnace through a 6-inch pipe connected to the city service mains. Air is supplied by an electrically driven blower.

The primary burners are operated with a smaller proportion of air to gas than is required for full combustion. The additional air needed to complete combustion and distribute the fire is supplied by secondary air inlets extending through the walls of the furnace at regular intervals toward the top, each inlet being provided with a regulating valve (Fig. 25). The air is so directed as to avoid impingement on the test column, and to set up a whirling motion of the furnace gases as an aid in securing uniform temperature distribution.

(*c*) OPERATING DETAILS

Means for regulating the gas and air supplied to the burners and the draft from the furnace chamber are located on the main floor An emergency cut-off valve with electrical control is placed in the gas main leading to the furnace, with switches for operating it set at several points in the building.

A Hoskins direct-reading millivolt-temperature indicator with connections to four chromel-alumel furnace thermocouples is placed near the furnace for use in regulating its temperature.

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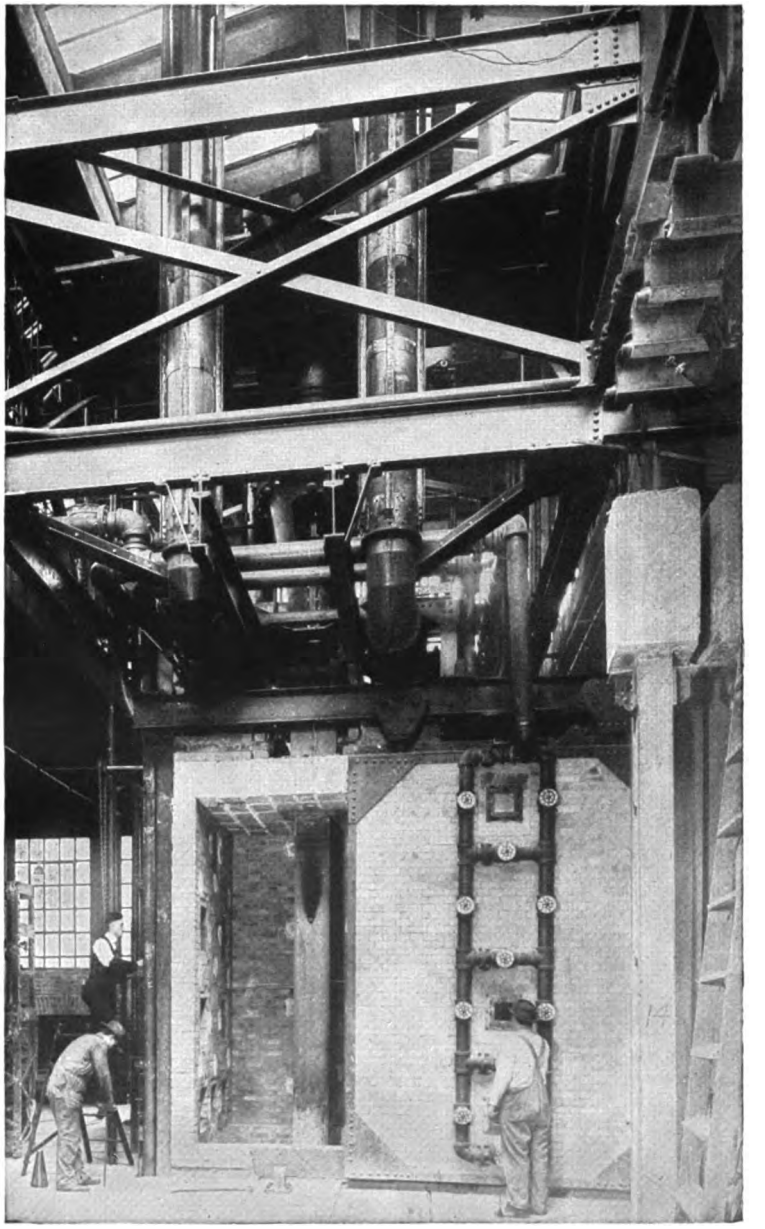

FIG. 26.—*General view of testing machine*

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A Gate to main cylinder
B Air outlet from auxiliary tank
C Water outlet from auxiliary tank
D Water outlet from main tanks
E Air outlet from main tank
F Water pump switch
G Air compressor switch
H Automatic pressure regulator
I Water inlet to auxiliary cylinder
J Water outlet from auxiliary cylinder.
K Air inlet to auxiliary tank
L Water inlet to auxiliary tank.
M Water pump pressure
N Water inlet to main tanks
O Air inlet to main tanks
P Air pump pressure
Q Auxiliary cylinder pressure.
R Main cylinder pressure.
S Pressure indicator
T Safety relief valve
U Auxiliary tank pressure.
W Main tank pressure.

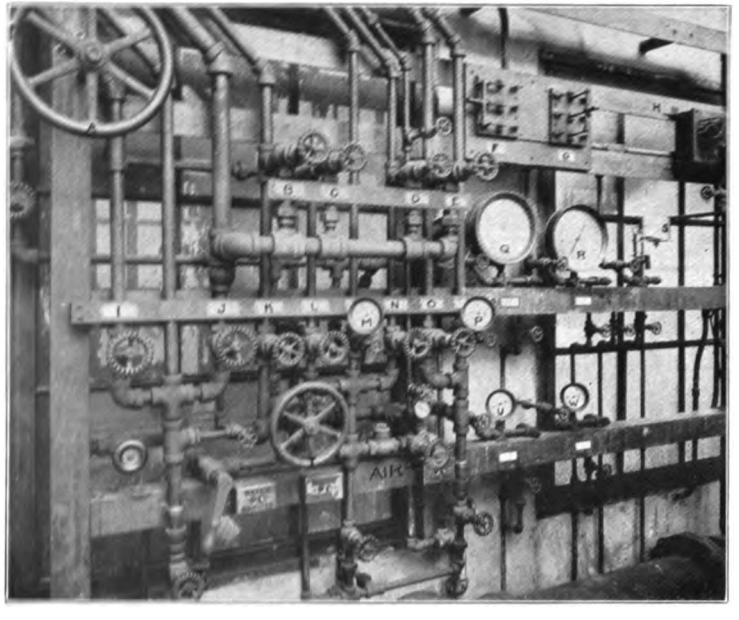

FIG. 27.—*Control board*

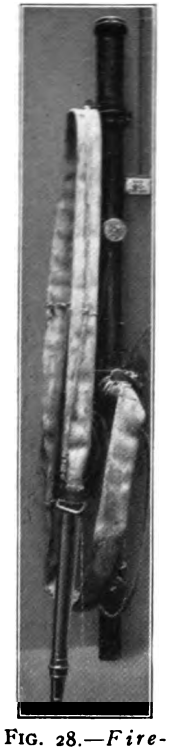

FIG. 28.—*Fire-stream apparatus*

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A—Gate to main cylinder.
B—Air outlet from auxiliary tank.
C—Water outlet from auxiliary tank.
D—Water outlet from main tanks.
E—Air outlet from main tanks.
F—Water pump switch.
G—Air compressor switch.
H—Automatic pressure regulator.
I—Water inlet to auxiliary cylinder.
J—Water outlet from auxiliary cylinder.
K—Air inlet to auxiliary tank.
L—Water inlet to auxiliary tank.
M—Water pump pressure.
N—Water inlet to main tanks.
O—Air inlet to main tanks.
P—Air pump pressure.
Q—Auxiliary cylinder pressure.
R—Main cylinder pressure.
S—Pressure indicator.
T—Spring relief valve.
U—Auxiliary tank pressure.
W—Main tank pressure.

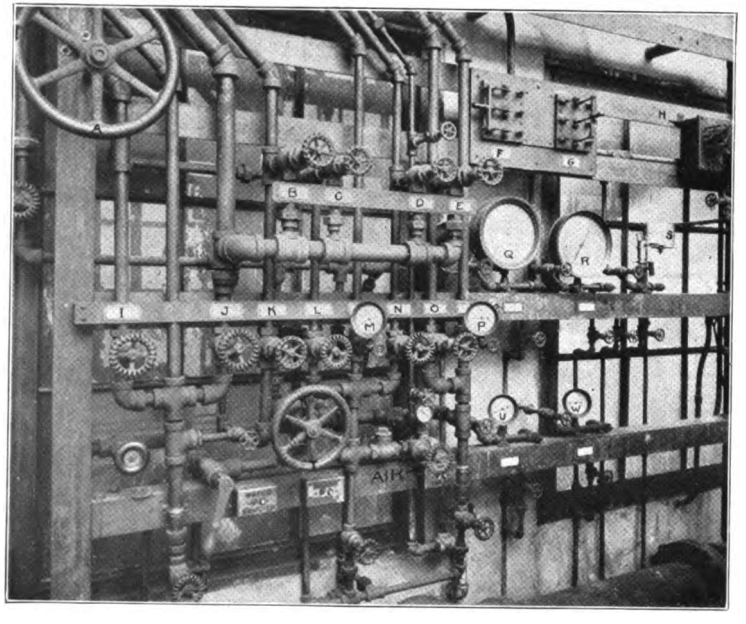

FIG. 27.—*Control board*

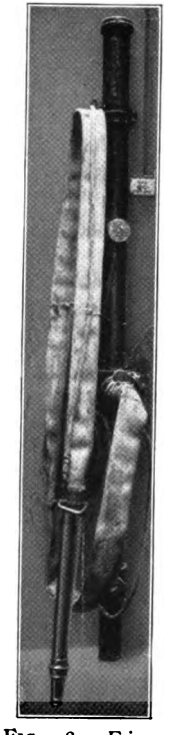

FIG. 28.—*Fire-stream apparatus*

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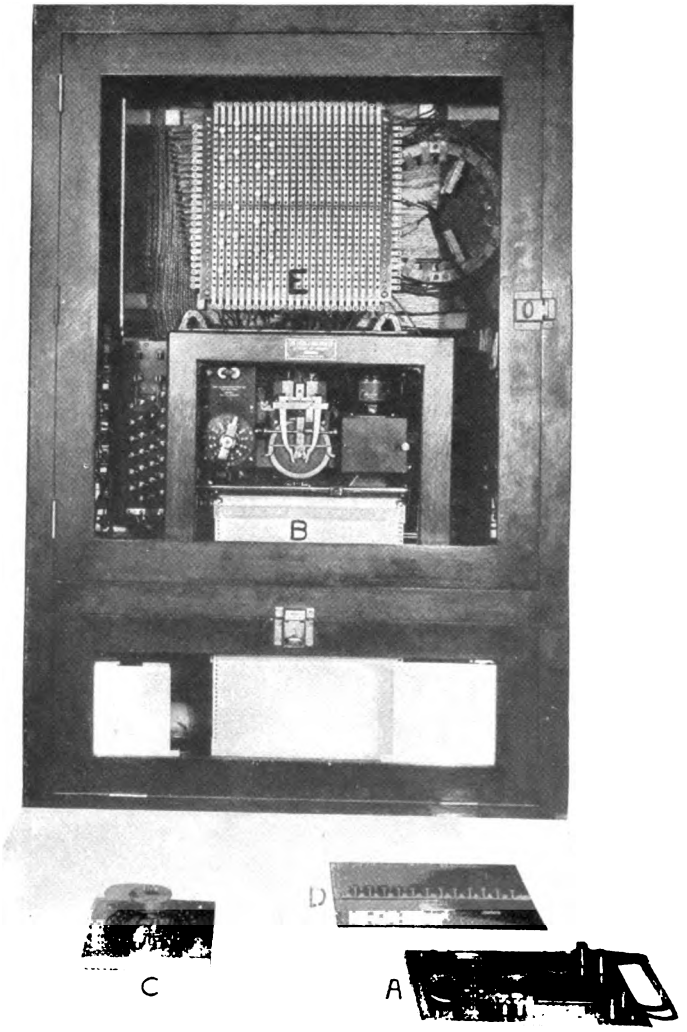

FIG. 29.—*Temperature-measuring instruments*

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5. FIRE-STREAM APPARATUS

The pressure head for the hose stream used in the fire and water tests is supplied by air pressure in a tank of 4500 gallons capacity. The water is applied through a standard Underwriter play pipe with nozzle 1⅛ inches in diameter (Fig. 28). It is connected with the hydrant through about 20 feet of 2½-inch cotton, rubber-lined hose. A throttling valve is placed at the outlet to the hose and a pressure gage connected with the base of the nozzle by means of which the desired pressure during water application is maintained.

The water was applied to three sides of the column, the nozzle being moved back and forth on one side of the furnace and kept at a distance of about 20 feet from the center of the column (Fig. 24).

VII. TEMPERATURE MEASUREMENTS

All temperature measurements were made by the thermo-electric method, platinum-platinum rhodium being used in the thermocouples placed in the furnace and base metal in those on the test columns. The electromotive force was measured with indicating and recording potentiometers, and the corresponding temperature obtained from comparison calibrations with standard thermocouples whose temperature-emf relations were known.

The temperatures are expressed on the Centigrade scale. The Fahrenheit equivalents are given at the right of the time-temperature plots. A centigrade and Fahrenheit conversion table is given in Appendix F (p. 375).

1. INSTRUMENTS

A view of the instruments used is given in Fig. 29, wherein an indicating potentiometer or indicator is shown at *A*, a recording potentiometer at *B*, dial and push-button switches at *C* and *D*, respectively, and bus-bar distributing board at *E*.

(a) INDICATING POTENTIOMETER

The potentiometer consists essentially of a means for securing a known variable electromotive force and balancing it against the electromotive force to be measured.

A wiring diagram of a simple form of this instrument is shown in Fig. 30. Current from a battery flows through the resistance

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ABCDE, of which the portion *BCD* is a slide wire. A scale is attached to the slider, giving the potential drop between points on the slide wire and the point *B* for a given value of the battery current. The latter is adjusted to the given value by opposing the electromotive force of a standard cell to the drop of potential over the resistance *AB* and varying the resistance *R* until the galvanometer included in the circuit shows no deflection. In a similar manner, an unknown electromotive force as that of a thermocouple, is balanced against the potential drop over some portion *BC* of the slide wire and its magnitude read from the attached scale.

Corrections for variation in the temperature of the cold end of the thermocouple require in general the addition of a small electromotive force to the observed one. With the potentiometer used

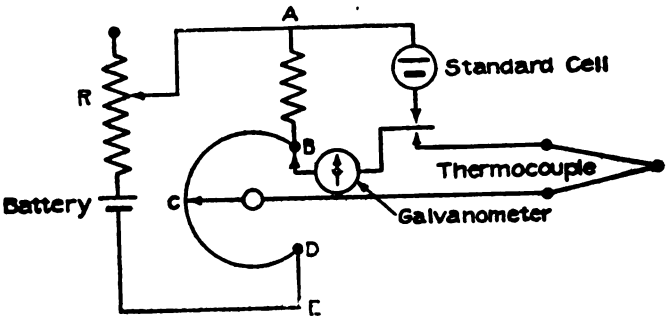

FIG. 30.—*Wiring diagram of indicating potentiometer*

in these tests, this could be accomplished mechanically at the time of making the observation by setting off the corresponding electromotive force on the cold-junction compensator, which has the same effect as moving the point *B* up the scale the same amount.

The potentiometer indicator used in these tests is equipped with two scales, one reading from 0 to 16 and the other from 0 to 80 millivolts. The accuracy of this particular instrument was found to be within one-fourth of 1 per cent of the total range at all points on the scale.

In the above method of measurement the indication is independent of the resistance of the thermocouple and lead wires, and no errors are introduced by variations in resistance caused by temperature changes in them.

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(b) RECORDING POTENTIOMETER

The wiring diagram of this instrument is an exact equivalent of that of the indicating potentiometer (Fig. 30), the electromotive force of the thermocouple being automatically balanced and recorded. It has scales of the same range as the indicating instrument, and a 13-point selector switch provides connections for a maximum of 12 thermocouples. On the remaining switch point the battery current is automatically adjusted to the correct value. Records are made at intervals of one minute. Where the temperature differences between the different couples connected are not too large, records can be made every half minute.

(c) ACCESSORIES

The accessories consist of a bus-bar board that was used for connecting any desired set of thermocouples with the recording potentiometer, a 12-point double-pole, double-throw, push-button switch, by means of which the couples connected with the recorder could be read on the indicator, and a 24-point double-pole dial switch for connecting thermocouples directly with the indicator.

2. FURNACE TEMPERATURES

Because of the high temperatures attained in column tests of long duration it was deemed advisable to use rare metal (Pt–90 Pt 10 Rh) thermocouples for the measurement of furnace temperatures.

(a) LOCATION OF FURNACE THERMOCOUPLES

In general, four couples were used to measure furnace temperatures, placed two on each of two levels at 3 feet and 9 feet above the fireproofing about the base of the column (Fig. 31). The couples on the 3-foot level passed through the furnace walls near the northwest and southeast corners; those at the 9-foot level near the northeast and southwest corners. Laterally the junctions of the furnace pyrometers were placed 15 inches from the nearest point on the column. The above symmetrical distribution is such that the center of gravity of the couple locations is identical with that of the portion of the furnace chamber above the fireproofing line at the base of the column.

The curves on the time-temperature plots (Appendix B) corresponding to the respective furnace couples are designated as *L–NW*, *L–SE*, *U–NE*, and *U–SW*, the letter preceding the dash indicating the level (*L*, lower; *U*, upper) on which the corresponding couple was placed; those following it, the corner nearest to which the couple entered the furnace.

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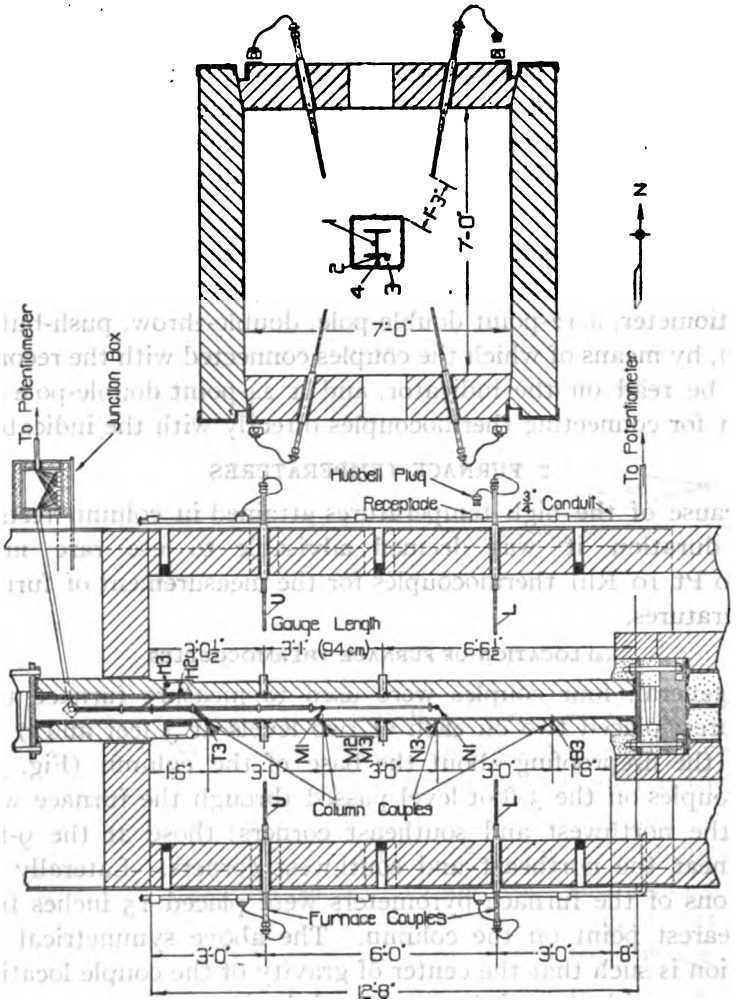

FIG. 31.—*Location of furnace and column thermocouples*

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The same system of designation is extended to include the few cases where couples were used in locations other than those mentioned. An exception is found in test No. 20, in the case of curves L_4 and U_4. The corresponding couples measured furnace temperatures at points 1½ inches from the column face, 3 feet and 9 feet, respectively, above the fireproofing line. They were made of No. 16 iron and constantan wires, with the junctions exposed directly to the furnace atmosphere.

(b) THERMOCOUPLE MOUNTINGS

Details of the thermocouple mounting are shown in Fig. 32. The use of rare-metal couples necessitated porcelain protecting tubes, which were covered with woven-asbestos sleeving to prevent breakage from sudden temperature changes. The alundum tube, with its winding of asbestos cord covered the outer 2 feet of the porcelain tube and served as a protecting sleeve where the mounting passed through the furnace walls.

The porcelain protecting tubes differed somewhat in wall thickness, as furnished by three different makers. The maximum and the minimum wall thickness were $\frac{5}{32}$ and $\frac{1}{16}$ of an inch, respectively. About 85 per cent of the total number of tubes used had a wall thickness of very nearly ⅛ of an inch.

Practically all of the woven asbestos sleeving was bought in two lots, both of the same nominal size. The first lot, comprising approximately ¾ of the sleeving used, ran about 15 feet per pound, the second lot about 24 feet per pound, as unwound from the spool.

(c) CONNECTIONS TO INSTRUMENTS

Permanent leads of No. 18 copper fixture wire were installed in a conduit leading from the case housing the recording potentiometer to the base of the furnace wall, from whence branches extended up to the thermocouple points. Hubbell receptacles were provided at the couple outlets (Fig. 31), which received the Hubbell plugs terminating the short leads attached to the cold ends of the furnace couples (Fig. 32). At the instruments the leads led to the bus-bar board, where each of the four furnace couples was connected to three recorder points, and also through the push-button switch with the indicator.

The temperatures of the cold junctions in the heads of the furnace pyrometers were measured by thermocouples formed of one of the copper leads in each head and No. 18 constantan wires

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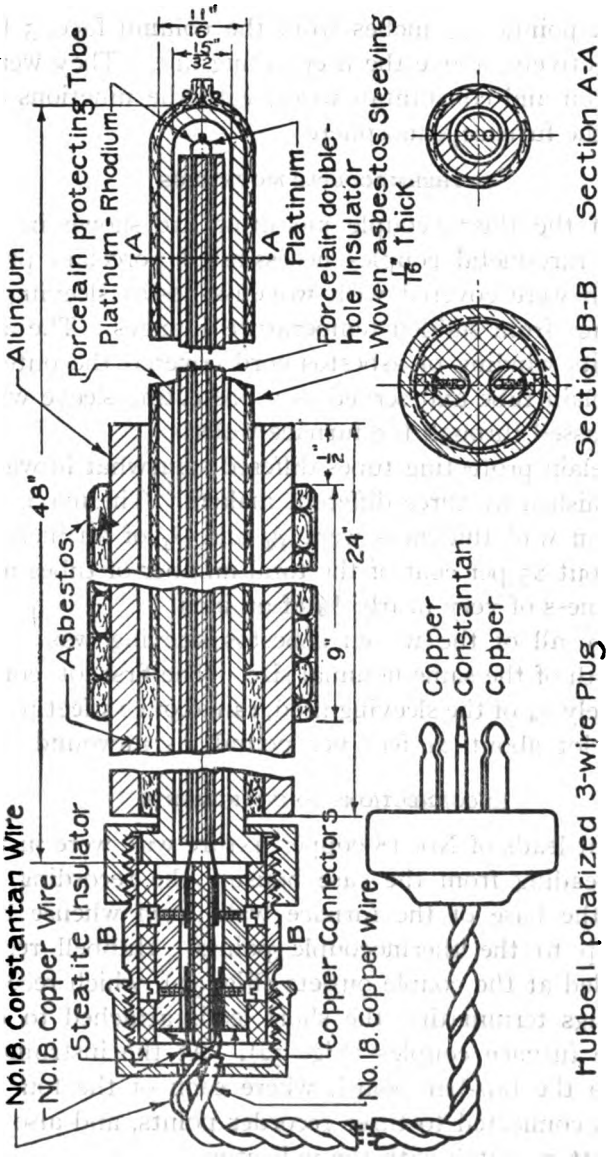

FIG. 32.—*Detail of furnace thermocouple mounting*

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(Fig. 32), which were installed in the same conduit as the copper leads, all wires extending to the case housing the recorder, from whence copper wires led through the dial switch to the indicator.

3. COLUMN TEMPERATURES

The base-metal thermocouples used for measuring column temperatures were of No. 16 (B. & S. gage) iron, No. 16 and No. 18 constantan, and No. 18 nickel wires, in the combinations, iron-constantan and nickel-constantan, 90 per cent of the total number of couples used being of the iron-constantan combination. As supplied, a portion of the wire carried a thin waterproofed asbestos covering, while the remainder was bare. As used on the column all wire was covered with closely fitting asbestos sleeving.

(a) ATTACHMENT OF THERMOCOUPLES TO COLUMN

A preliminary test consisting in subjecting to heat in a gas-fired furnace two concrete-covered lengths of wrought pipe, with thermocouples placed on the metal and in the covering, indicated that the proposed method of measuring column temperatures gave, in general, reliable results. Two methods of attaching the couples to the pipe section were tried out and appreciable differences of indication found to be caused by the given variation in this detail. By the one method, which will be termed "peening" the ends of the individual couple wires were placed in closely-fitting holes drilled in the metal and secured in place by driving in the metal around the holes, using a slotted punch placed in turn over each wire. Couples attached by the other method, and referred to as "insulated," had fused or twisted junctions that were laid against a piece of mica about 0.01 inch thick placed between it and the metal.

Since it was apparent that the peened couples gave more nearly the true temperature of the metal, the couples on the columns prepared subsequent to the preliminary test were attached by peening, except that one or two insulated couples were added at certain important locations as a check on the adjacent peened couples.

The couples placed before the preliminary test was made were all insulated and include those under the concrete covering or filling of all partly protected columns, test Nos. 14 to 22, inclusive, and the couples of the concrete-protected column of test No. 28 and those on the vertical bars of the reinforced-concrete column of test No. 73.

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To decrease the danger of injury to the peened junctions in placing the coverings and in testing the columns, the region immediately surrounding the point of attachment was covered to a thickness of about ⅛ of an inch with asbestos wool loosely placed under a shield of sheet iron 1 inch square and 0.015 inch thick. The couple wires, protected by the applied sleeving, were held firmly in place under a strap-iron clip screwed to the column near the junction. The same clip also held the shield over the asbestos wool in place. Similar clips, placed about 18 inches apart, held the couple wires on the column up to 1 foot below the top bearing, where they were all led out through a 1¼-inch pipe (Fig. 31).

The asbestos wool and shield were omitted over the insulated junctions, since it was thought that they would not be subjected to as large strains as the junctions that were peened.

As it was very difficult to attach couple junctions to the inside of hollow steel or cast-iron columns by the peening method, the junctions were laid against the metal and protected with asbestos wool, as described above.

In placing couples in the column coverings or in locations other than on the metal, suitable means were used for supporting them in place, the methods varying with the conditions presented by the different types of columns and coverings. In all cases the details were so arranged as to reduce to a minimum heat interchange between the couple junction and its support as well as between the junction and the couple wires. The latter was accomplished by placing a sufficient length of the wires immediately leading from the junction in a plane where, with the contemplated fire exposure, the same temperatures would normally obtain as at the junction.

(*b*) LOCATION OF COLUMN THERMOCOUPLES

In all tests, except in those of timber columns, the thermocouples were spaced vertically in four general locations, *B*, *N*, *M*, and *T*, 1 foot 6 inches, 4 feet 6 inches, 7 feet 6 inches, and 10 feet 6 inches, respectively, above the fireproofing line at the base of the column (Fig. 31). The lateral locations are numbered 1, 2, 3, 4, etc., and marked by small solid circles on the sectional location diagrams of the time-temperature plots in Appendix B. The letter and the number designating, respectively, the vertical and horizontal position, as combined in the curve designations, define completely the location of the couple.

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Additional couples, designated on the plots (Appendix B) as *H2* and *H3*, were placed on some of the structural-steel and cast-iron columns at the base and on the edge of the beam bracket near the top (Fig. 31).

In general the letter *i* following the numbers in the column couple designations (Appendix B) is to be interpreted as "*insulated*," referring to the method of attachment. Exceptions occur only in test Nos. 3 and 4 (Fig. 90) in the case of the couples attached on the inside of the column.

Letters indicating points of the compass are, in general, included in the designations of the couples that are not on the side of the column on which the general vertical distribution occurs. In the latter case, the couples are designated by the location letter and number only. Exceptions to the above are designations for the insulated couples in test No. 27 (Fig. 98), No. 47 (Fig. 101), and Nos. 62 and 63 (Fig. 126), which are located near the couple on the same level that has a direction letter in its designation.

The couples whose curves on the plots for the timber column tests (Figs. 140 and 141) carry the designation *T* were located 13 inches below the cap bearing, the lateral positions being evident from the location diagrams, as are also the locations of the couples on the metal of the caps and pintles. The other vertical positions were the same as in the other tests.

In tests Nos. 10 and 11 (Fig. 92) and No. 13 (Fig. 93) the prime mark (′) on one couple designation in each test indicates that the given couple had no asbestos and sheet-iron protection over it.

In test No. 21 (Fig. 96) the prime marks on two curves indicate couples whose leads for 19 inches immediately above the junction were protected by strips of asbestos board 1 inch wide and ¼ inch thick, the object being to determine what influence possible heat interchange between the leads and the junction would have on the indication of the couple.

(c) CONNECTIONS TO INSTRUMENTS

The wires of the column thermocouples led from the outlet in the column head above the furnace roof to the junction box placed to one side of the latter (Fig. 31), where they were put under binding posts on a set of connecting blocks. Duplex leads of insulated iron and constantan wire were permanently

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installed from the junction box to the bus-bar board in the case housing the recording potentiometer, from whence copper leads extended to the dial switch. The column couples could accordingly be connected with either the indicator or the recorder.

4. METHOD OF TAKING AND REDUCING OBSERVATIONS

In most tests the rate of temperature change was not more rapid, but that readings of both furnace and column thermocouples could be taken on the indicating potentiometer. Since it was found that the observations taken with it were more readily reduced to final form than the recorder records, the temperature determinations as given in the tables and plots are in general based on indicator readings, although the recorder was connected with the furnace couples in practically all tests, and its records served as a general check on temperatures during the test, and were also used in making occasional interpolations between indicator readings in plotting the results.

In taking observations with the indicating potentiometer the electromotive forces of the furnace or the column couples were read in succession in a chosen order, then repeated in reverse order, with equal time intervals (10 seconds or less) between consecutive readings. Mean values were taken of the direct and reverse readings on the respective couples, which, on the assumption of linear temperature change, are equivalent to simultaneous readings on all couples. The readings of the copper-constantan couples at the cold junctions of the furnace pyrometers, were reduced to the corresponding platinum-platinum rhodium electromotive forces, and added to the indications of their respective furnace couples. In the case of the iron-constantan column couples the cold junction correction, which was due to temperature differences between the junctions at the bus-bar board and the zero of the calibration plot, were set off directly on the compensator. For the nickel-constantan column couples a further correction was applied due to the electromotive force introduced by the couple formed in connecting the nickel couple wire to the iron wire leading from the junction box above the furnace to the instruments.

After the proper corrections to the observed electromotive forces had been applied the corresponding temperatures were obtained from the calibration plots.

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5. CALIBRATION OF THERMOCOUPLES

(a) FURNACE THERMOCOUPLES

The furnace couples, in addition to being compared several times during the series of tests with a standard couple, were frequently submitted to a homogeneity test. The latter is, in effect, a determination of the electromotive force, at frequent points along the couple wire, against standard wire of the same kind. This test was necessary, since the electromotive force of a nonhomogeneous couple—that is, one whose thermoelectric properties vary from point to point along the wire—depends not only on the temperature of the junction, but also on the magnitude and position of the temperature gradients along the length of the couple, which, in general, were not the same as placed in the column-testing furnace as in the furnace used for calibrations.

Changes in calibration were found to be very small as long as the protecting tubes served their intended purpose of excluding the furnace gases. When a protecting tube was broken, or the couple wires had in any manner been exposed to the furnace gases, a homogeneity test was made. When such test showed no electromotive force greater than the equivalent of 15° C at 1000° C, the couple was used again without further treatment. When this value was exceeded, the wire was either brought sufficiently near to its original (uncontaminated) condition by annealing for several hours at 1500° C by passing an electric current through it, or the bad portion of the wire was removed and good wire substituted.

An accuracy within 1 per cent of the indicated temperatures was attained in the great majority of tests. In the case of individual couples in a few tests the homogeneity determination indicated possibility of errors as high as 5 per cent, although the actual errors incurred were probably considerably smaller, since the nonhomogeneous portion of the couple was not necessarily in the region of maximum temperature gradient.

(b) COLUMN THERMOCOUPLES

Practically all of the wire used in constructing the column couples was bought in two lots, one of which was purchased in 1914, the other in 1917.

A preliminary study of the variations in the thermoelectric properties among a number of samples taken from the wire in

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the first lot indicated that to secure the desired accuracy calibration of individual couples made from this wire would be necessary and that differences between the indications of individual couples were approximately proportional to the temperature.

The latter result showed that a good estimate could be made of the calibration of a given couple by comparing it at one temperature with a standard, and this practice was, in general, followed for all the couples made from the wire in the lot first purchased. Tests on about 50 representative couples made from the wire in the second lot showed sufficiently small variations among the individual couples as to permit the use of one temperature-emf curve for all the couples made from this lot.

The results of the large number of tests made in studying the wire used in constructing column couples indicate that the accuracy was within 2 per cent of the indicated temperatures.

VIII. DEFORMATION MEASUREMENTS

1. GENERAL OUTLINE

Included under this head are means for measuring (1) the unit compression and expansion over a definite gage length, (2) the total depression or expansion of the column measured at a point above its heated portion, (3) the lateral center deflection.

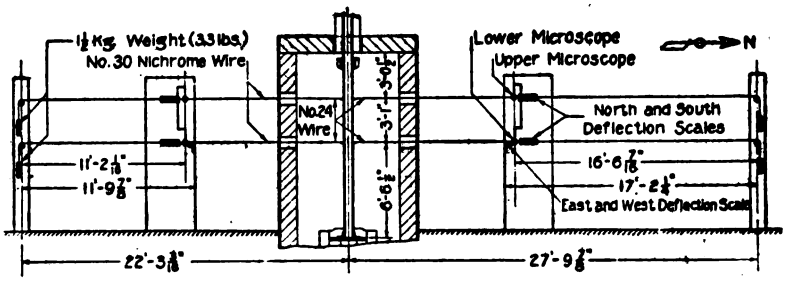

FIG. 33.—*Diagram showing method used for measuring deformation*

The general method used for measuring unit deformation and center deflection is shown in diagram in Fig. 33 and a view of the furnace with attachments in place is given in Fig. 34. Fine nickel-chromium alloy wires were connected to opposite sides of the test column at points symmetrical with its center, using for each a short length of heavier wire and a threaded insert. The

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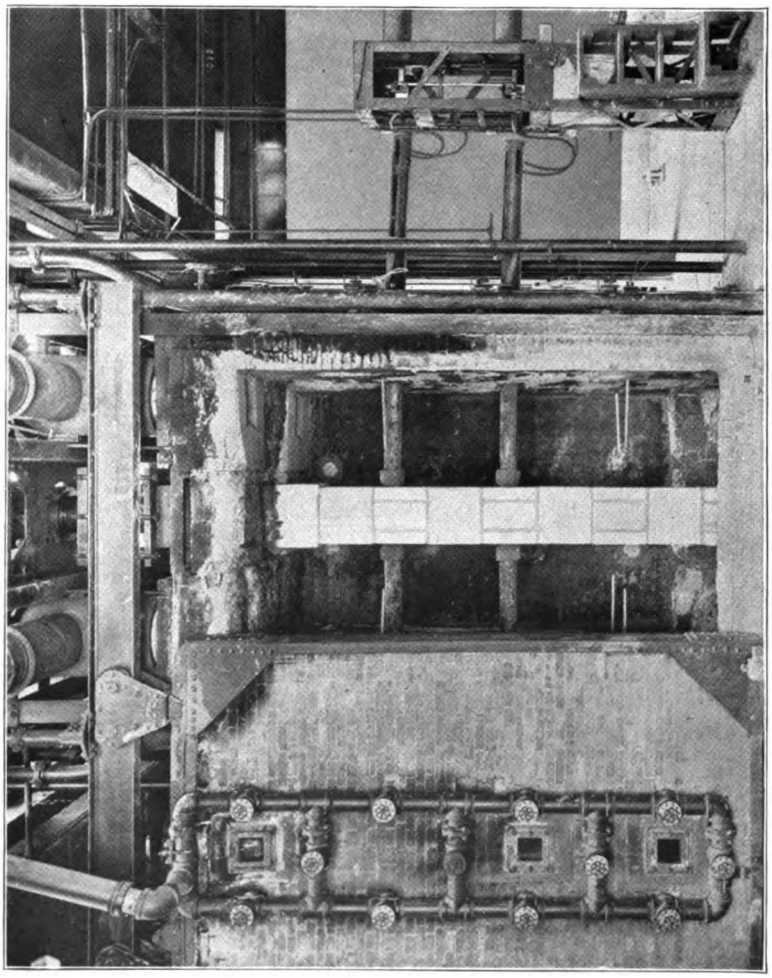

FIG. 34.—*Testing furnace with furnace couples, column, and deformeter in place*

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other ends of the wires were weighted and passed over small flanged wheels located in the end posts. The movement of the wires was measured at an intermediate station on each side of the furnace.

(*a*) ATTACHMENT AND PROTECTION OF WIRES

A detail of the insert used for attaching the wire to the column is given in Fig. 35, as are also details of the water-cooled insulating tube. The hole for the insert was prepared before the column was covered or placed in the furnace and a 1¼-inch pipe with collar was bolted around it to provide a clear space for the wire. The pipe projected outside of the covering to afford support for the insulating tube. With the column in place in the furnace, the insert, with wire attached and held in a suitably formed tool, was threaded into the column flange and turned so that the wire was left supported on the upper ledge of the insert.

The insulating tube was designed to protect the wires from the heat of the furnace, as otherwise they would break under the tension to which they were subjected. The circulating water was contained in the annular space between the 2 and 3 inch wrought pipes, the outer pipe being further protected by fire-clay tubing. A 5-inch wrought pipe, with the annular space between it and the 3-inch pipe filled with finely crushed fire brick, was substituted for the fire-clay tubing in the later tests of the series.

Outside of the furnace the wires were shielded by sheet-metal tubes.

2. UNIT COMPRESSION AND EXPANSION

(*a*) MICROMETERS AND MOUNTING

The vertical movement of the wires was measured by means of microscopes set in micrometer slides (Fig. 36). The latter were mounted one on each end of nickel-steel bars supported near the middle. The supports were provided with pivots and slides to secure angular, vertical, and horizontal adjustment of the bar as a whole. The micrometer heads are graduated to 0.005 mm and can be read to the nearest 0.001 mm, the total range of the slide being 50 mm.

(*b*) METHOD OF TAKING AND REDUCING OBSERVATIONS

Readings were taken at the same time on both sides of the furnace, the lower micrometer, upper micrometer, and again, the

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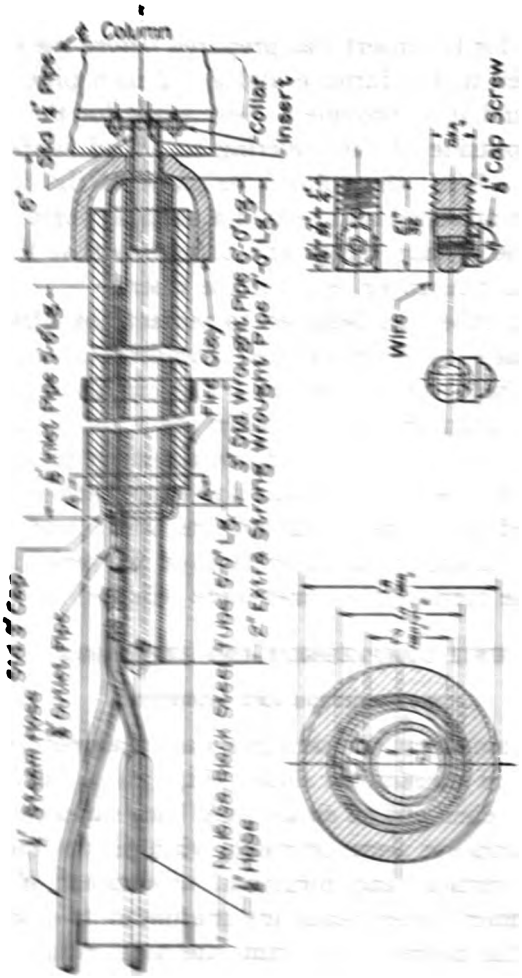

FIG. 14.—Detail of insulating plug and insert

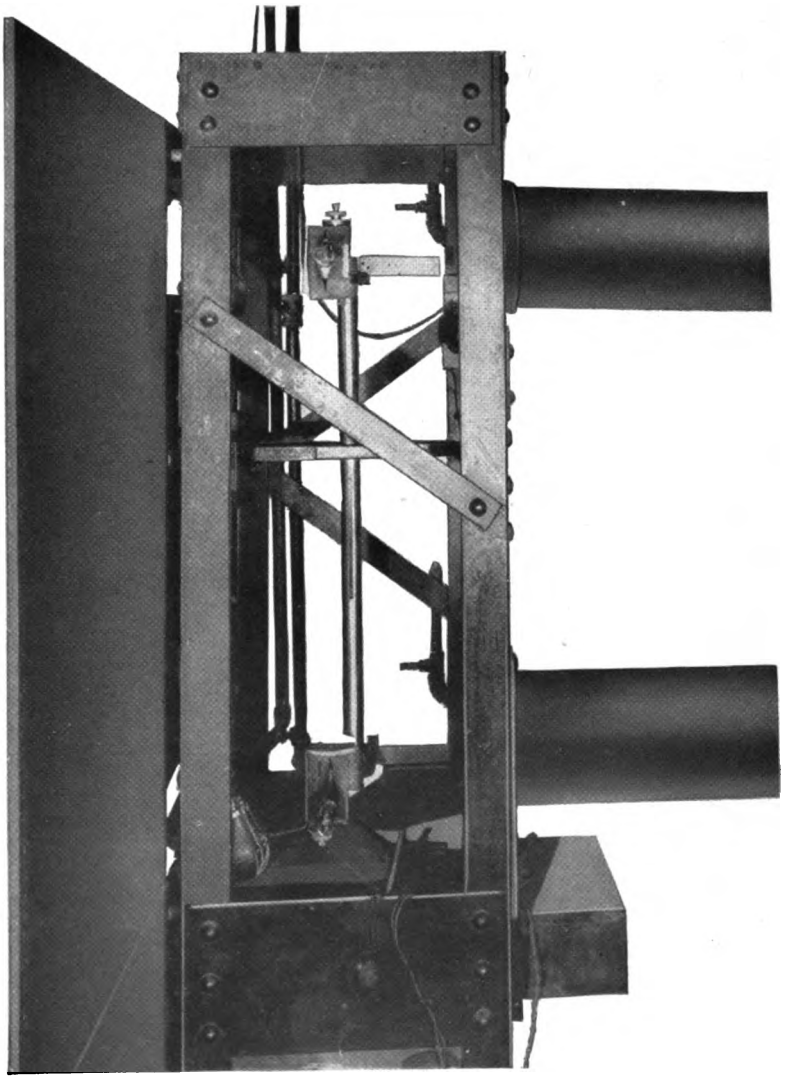

FIG. 36.—*Apparatus for measuring deformation*

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lower micrometer being read in the given order at equal intervals. The compression or expansion in the gage length was obtained by multiplying the movement of the upper wire relative to the lower wire at the point of observation by the ratio the distance from the column to the wire support in the end post has to the distance from the latter point to the microscope (Fig. 33). The unit deformation is the average of the values of total deformation obtained as given above, divided by the distance between gage points (94 cm).

3. CENTER DEFLECTION

The lower gage point was located near the center of the heated length of the column, and the center lateral deflection was measured by means of polished nickel scales, readings being taken with reference to points on the wires and their reflection in the scales.

The east-and-west deflection scales were placed perpendicular to the wires, and the deflection of the column was obtained by multiplying the observed movement at the scale by the ratio of distances, as given in Fig. 33, of the column and of the scale, respectively, from the wire supports in the end posts, taking the average of values obtained on the two sides.

The north-and-south deflection scales were placed parallel with the wires, small flat disks being attached to the latter in front of the scales, with reference to which readings were taken. The deflection in the north or south direction is the average of the movements observed at the two posts, assuming the expansion of the measuring wires on both sides of the column to be equal.

4. TOTAL DEPRESSION OR EXPANSION

The total vertical depression or expansion of the column before failure, as indicated by the movement of the head of the loading ram, was determined for the columns in the fire and water test series, the timber columns, and a few subsequent tests of structural-steel and cast-iron columns. For this purpose a single No. 30 wire on one side of the furnace only was attached to the bearing below the head of the ram plunger, the details of attachment and measurement of its movement being the same as described in paragraph 2 of this section.

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5. CALIBRATION AND ACCURACY

Calibrations of the instruments and appliances used for measuring unit deformation and study of the test results, indicate possible maximum errors of 20 parts in 100 000 (0.0002), which correspond to errors in instrument readings of about 0.008 inch (0.2 mm). These errors were caused mainly by the change in sag of the wires due to moisture in the insulating tubes and also by movement of the column as a whole in the time interval required by a set of micrometer readings. The errors due to irregularities in micrometer screws and slides and in parallelism of slides and of microscopes as mounted on the supporting bar were relatively small.

In determining the total expansion or depression of the column by measurement of the movement of the loading ram a large error was incurred in applying the load due to deflection of supports. During the subsequent fire exposure under constant load the supports appear to have remained fairly rigid, the maximum errors for this period being probably within 0.01 inch (0.25 mm) for the steel and cast-iron columns and 0.04 inch (1 mm) for the timber columns.

The center deflection in the east-and-west direction was determined with an accuracy of about 0.04 inch (1 mm) and in the north-and-south direction within about 0.08 inch (2 mm). The principal source of error in the latter case was unequal expansion of the wires due to unequal temperatures within the insulating tubes.

IX. METHOD OF TESTING

The columns in the fire-test series were subjected to a constant working load and fire exposure increasing according to a predetermined time-temperature relation until failure occurred or until they had withstood the test eight hours, or more.

In the fire and water tests the working load was maintained constant and the column exposed to fire for a predetermined period when water at given pressures was applied by means of a hose stream.

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1. LOADING FORMULAS AND APPLIED LOADS

(a) WORKING LOADS

The formulas used in calculating the working loads to be placed on the columns were among those in most general use at the time the method of procedure for these tests was determined. They are given in Table 41, where are also given the properties of the columns and the calculated and applied working loads.

The applied loads, which include weights of the column head and bearing blocks (2500 pounds), exceed the calculated working loads

TABLE 41.—COMPUTED AND APPLIED WORKING LOADS

SECTION	Formula	Nominal Area, Sq. In.	l/r	Computed Load		Applied Load	
				Lb. per Sq. In.	Total, Lb.	Total, Lb.	Per cent of Computed Load
Rolled H	16000-70 l/r	10.17	75.6	10710	108900	112500	109.7
Plate and Angle	do	13.09	111.8	8170	106200	116000	109.2
Plate and Channel	do	8.76	64.7	11470	100500	111600	110.5
Latticed Channel	do	7.78	44.0	12930	100500	111600	110.5
Z-bar and Plate	do	9.32	81.7	10220	95200	105000	109.5
I-beam and Channel	do	10.12	72.1	12930	110800	122000	110.1
Latticed Angle	do	8.44	40.7	13160	111000	122500	110.4
Starred Angle	do	13.27	108.5	8435	111900	124000	110.7
Round Cast Iron (Vertically Cast)	10000-60 l/r	14.45	63.2	6210	89700	98500	109.8
Round Cast Iron (Horizontally Cast)	do	14.73	63.2	5910	87100	95500	109.6
Round Cast Iron (Concrete filled)	do	14.73	63.2	5910	87100	95500	109.6
			l/d				
Steel Pipe (Concrete filled)	As (13500-140 l/d) +Ac (1600-11 l/d).	Steel 6.93 Concrete 38.74	19.6	Steel 10750 Concrete 784	104900	114500	109.2
Reinforced Steel Pipe (Starred angles embedded in the concrete filling)	do	Steel 18.36 Concrete 40.07	17.7	Steel 11060 Concrete 805	234700	236000	100.6
Square Vertically Reinforced Concrete	450 (Ac+15 As)	Concrete 140 Steel 4.00	12.7	Concrete 450 Steel 6750	90000	101000	112.2
Round Vertically Reinforced Concrete	450 (Ac+15 As)	Concrete 127 Steel 6.00	11.7	Concrete 450 Steel 6750	97500	107500	110.3
Hooped Reinforced Concrete	650 (Ac+15 As)	Concrete 129 Steel 3.38	11.7	Concrete 650 Steel 9750	117000	129000	110.3
Timber	$1000\left(1-\frac{1}{80d}\right)$	129.4	13.4	833	107700	118500	110.0

l—Effective length, inches.
r—Least radius of gyration, inches.
d—Diameter or side, inches.
As—Area of vertical steel, sq. inches.
Ac—Area of concrete, sq. inches.

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by about 10 per cent for most of the columns. The principal part of the excess was incurred in the first tests of the series which were made before the calibration of the loading ram was completed. It was decided to maintain the same loads for the subsequent tests as representative of a moderate condition of overload.

No allowance was made for the load-carrying capacity of the covering or filling in any of the tests except in case of the pipe columns where the loads were calculated according to the loading formula in use by the manufacturer who supplied the columns.

(*b*) LOADING TO FAILURE

In case the column withstood the 8-hour fire test it was immediately loaded to failure under full fire exposure.

In the fire and water test series, three protected structural steel, the two unprotected cast-iron, and the three reinforced-concrete columns were loaded to failure after they had cooled. Four protected structural-steel columns after subjection to fire and water application, were loaded to a little over twice the applied working load and reserved for use in further tests.

2. FIRE EXPOSURE

(*a*) CHARACTER OF THE FIRE

The fuel used was carbureted water gas from the city service mains. It was admitted at the primary burners with insufficient air for immediate complete combustion, in order to avoid excessive temperatures at the bottom of the furnace and allow further combustion to take place at the secondary air inlets. The gas was generally burned with sufficient air to prevent deposit of soot on the test column or furnace walls.

(*b*) PRELIMINARY PANEL TESTS

During the first half hour it is not in general possible to regulate the temperature of a large testing furnace to correspond closely with predetermined temperatures; also, the determination of temperature is complicated by the lag of the furnace pyrometers with the rapid temperature rise occurring during this period. To obtain information on the effect of varying the rate of temperature rise, two panels built up of the five kinds of hollow clay tile used as protection in the column tests, were subjected to fire on one side for one-half hour and allowed to cool in place. The

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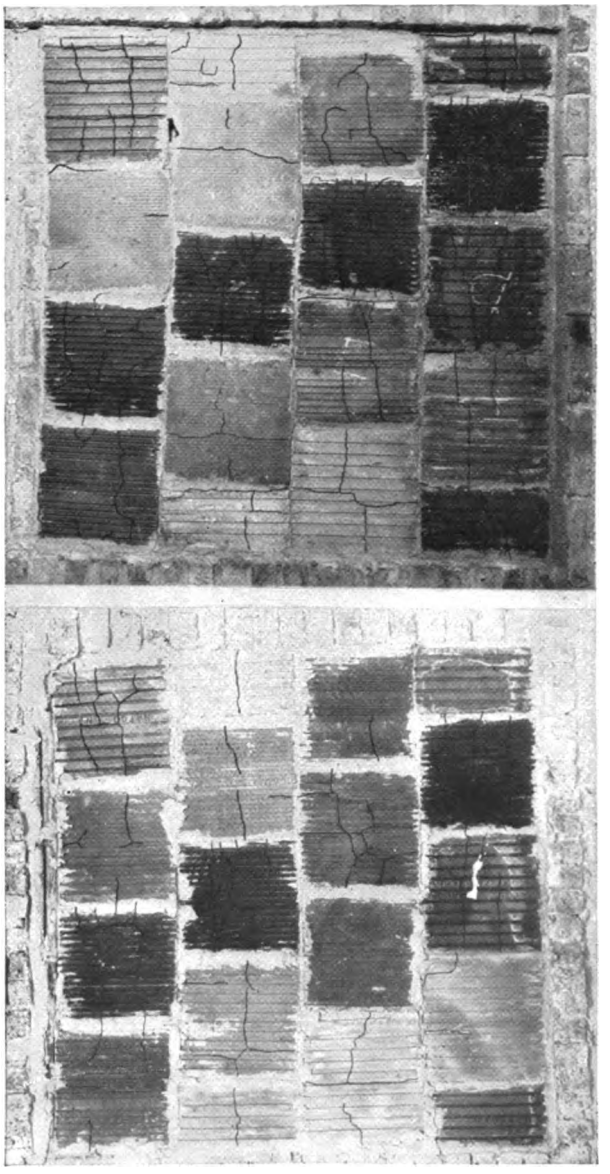

Panel No. 2

Panel No. 1

Black lines trace fine cracks developed in fire test

FIG. 38.—*Preliminary tile panels after test*

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average furnace temperatures obtaining in the two tests are given in Fig. 37, and in Fig. 38 the condition of the panels after test is shown. Most of the tile developed fine cracks which extended to the inside of the inner shell and in a few cases through the cross webs to the outer shell. Cracks along the cross webs near the inner shells were common and some inner shells were loose. While in one test, the indicated furnace temperature rise in the first 10 minutes was over twice as rapid as in the other, the condition of the two panels after test was nearly the same.

(c) TIME-TEMPERATURE CURVE

The temperature of the furnace was regulated to conform as nearly as practicable with a predetermined time-temperature reference curve shown in broken lines in Fig. 39. Here is also shown the furnace temperature attained, the curve being obtained by

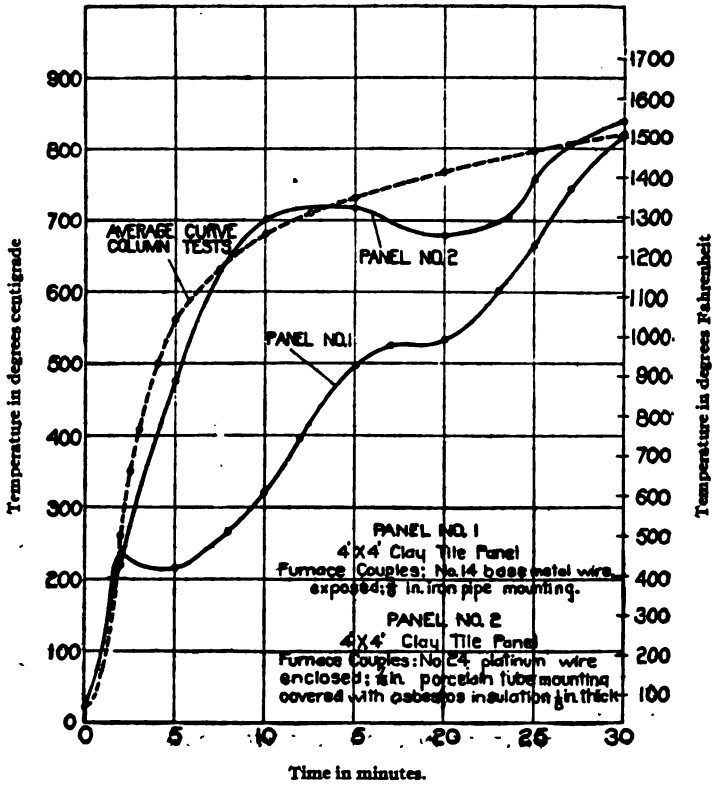

FIG. 37.—*Furnace temperatures, preliminary panel tests*

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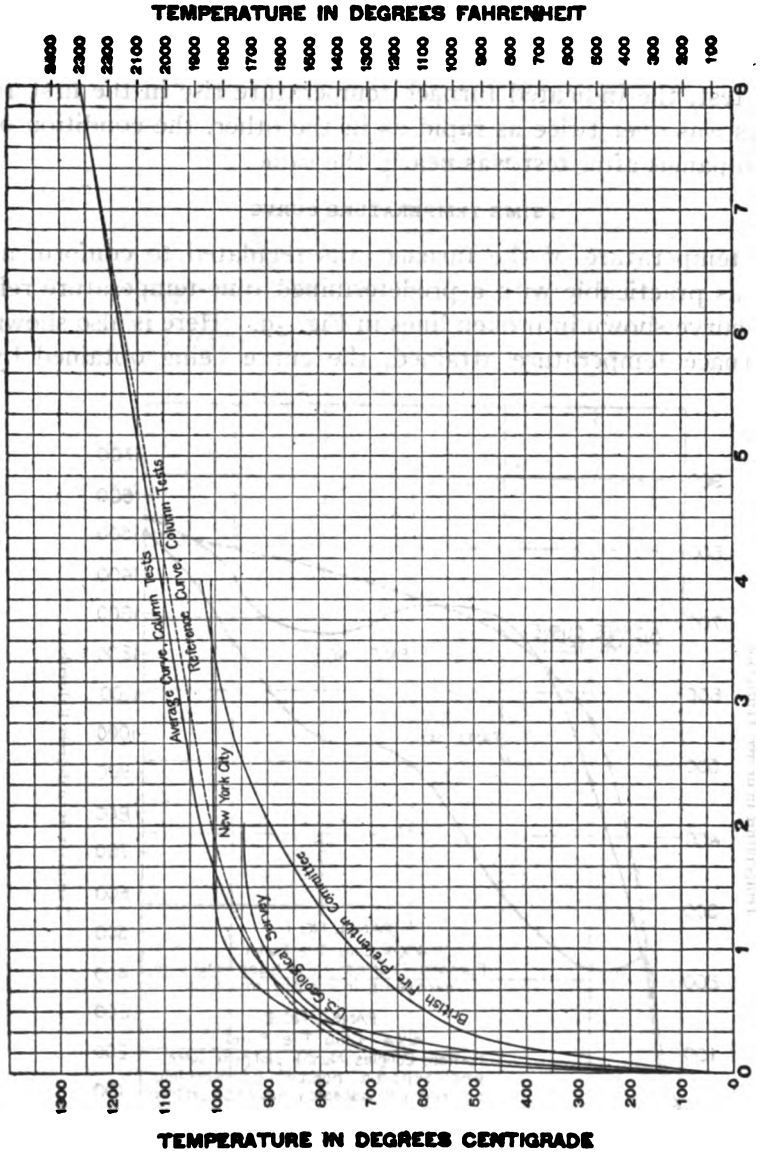

FIG. 39.—*Comparison of furnace temperatures used in fire tests*

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averaging the results from the full number of tests. Three curves giving the average indicated furnace temperatures attained in previous fire tests are also given.[2]

On the reference curve of the column tests a furnace temperature of 832° C (1550° F) obtains at the end of 30 minutes, and 927° C (1700° F) at the end of the first hour. From this point the temperature increases at a gradually varying rate up to 2 hours, when the temperature is 1010° C (1850° F). Subsequent to this the rate of rise is constant, 1101° C (2000° F) being attained at the end of 4 hours, and 1260° C (2300° F) at the end of 8 hours. The average indicated furnace temperature was somewhat below the reference curve in the first hour and above it by a maximum of 25° C during the subsequent periods.

(*d*) INFLUENCE OF PYROMETER MOUNTING ON INDICATED TEMPERATURES

With the type of mounting it was deemed necessary to use in the column tests (Fig. 32) the pyrometers did not indicate the full measure of the temperature effects within the furnace chamber, due mainly to the heat insulating and radiating properties of the mounting.

(1) TEMPERATURE LAG.—The retardation due to the heat insulation given the thermocouple by the mounting was particularly marked during the first part of the test period, when, with the obtaining rapid temperature rise, the indicated temperatures were considerably lower than those acting on the mounting on account of the time required to produce temperature equilibrium throughout the latter. This effect will be termed "lag," and a measure of its extent can be obtained by application of the principles governing heat interchange.

When the rate of temperature rise is not too rapid and the heat interchange is chiefly due to conduction, within a given range, the temperature lag is very nearly proportional to the rate of rise of indicated temperature, or,

$$U - \theta = \lambda \frac{d\theta}{dt} \qquad (1)$$

where θ is the temperature indicated by the pyrometer, U is the

[2] cf. Woolson and Miller, Paper No. 27a, International Association for Testing Materials, 1915; floor test Nos. 55 to 74. Humphrey, U. S. G. S. Bulletin No. 370; panel test Nos. 1 to 30. British Fire Prevention Committee, Journal No. 6, 1911; average from eight full-protection, Class B, floor tests.

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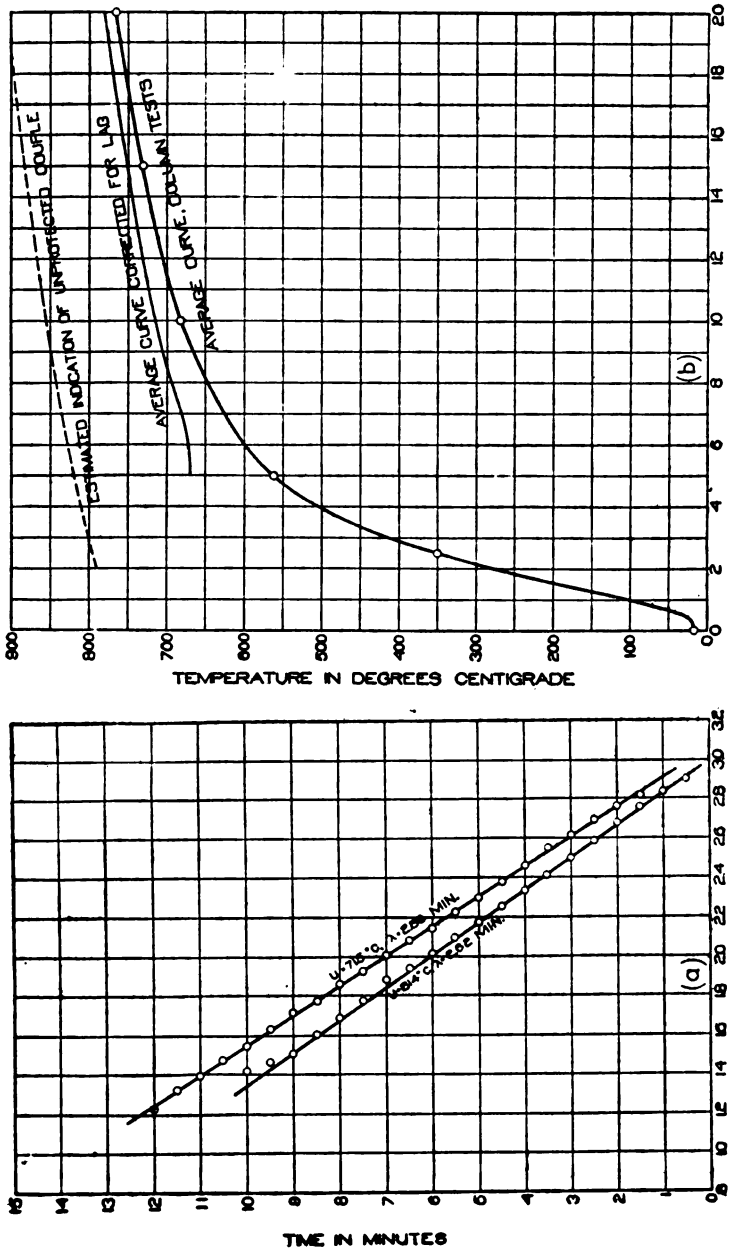

FIG. 40.—*Lag correction for furnace pyrometers*

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temperature the pyrometer would indicate if the furnace conditions at the given instant were maintained constant a sufficiently long time, $\frac{d\theta}{dt}$ is the rate of rise of indicated temperature, and λ is a constant for the pyrometer considered within a given range in U. To determine λ, the pyrometer was plunged into a gas-fired furnace whose temperature was maintained as nearly constant as possible and readings taken until the indicated temperatures showed little or no change. The experiment was begun as soon after the gas was lighted as was consistent with maintaining steady temperature in order that the furnace conditions might approximate those present during the first part of a column test. If U, for the conditions of the experiment, be assumed constant, from equation (1)

$$\lambda = \frac{t}{\log e\,(U-\theta)} + C, \qquad (2)$$

where t is the time after the pyrometer was introduced into the furnace and C is the constant of integration, which latter, in the method applied, is eliminated by subtraction.

The results of two determinations are given in Fig. 40 (*a*), λ being obtained from the slope of the line connecting the experimental points, multiplied by 0.4343, the modulus of the common system of logarithms. As applied for correcting the average curve of indicated temperature of the column tests, a lower value than the average experimental result was used to allow for impaired and thinner insulation that was present in a number of tests, a value of λ of 2.4 minutes being used up to indicated temperatures of about 800° C. Determinations made at higher temperatures gave lower results, values of about 2 minutes obtaining for mountings with unimpaired insulation tested at 950 to 1000° C.

In Fig. 40 (*b*) the lower curve is the average curve of indicated furnace temperature of the column tests for the first 20-minute period. Corrections for lag were deduced for the indicated temperatures given by the curve in the interval 5 to 20 minutes by substituting the given value of λ in equation (1), the rate of rise, $\frac{d\theta}{dt}$, being obtained from tangents drawn to the average curve at points ½ to 1 minute apart. As given by the difference between the original and corrected curves, the lag decreases from a little over 100° C at 5 minutes to 14° C at 20 minutes. From there on

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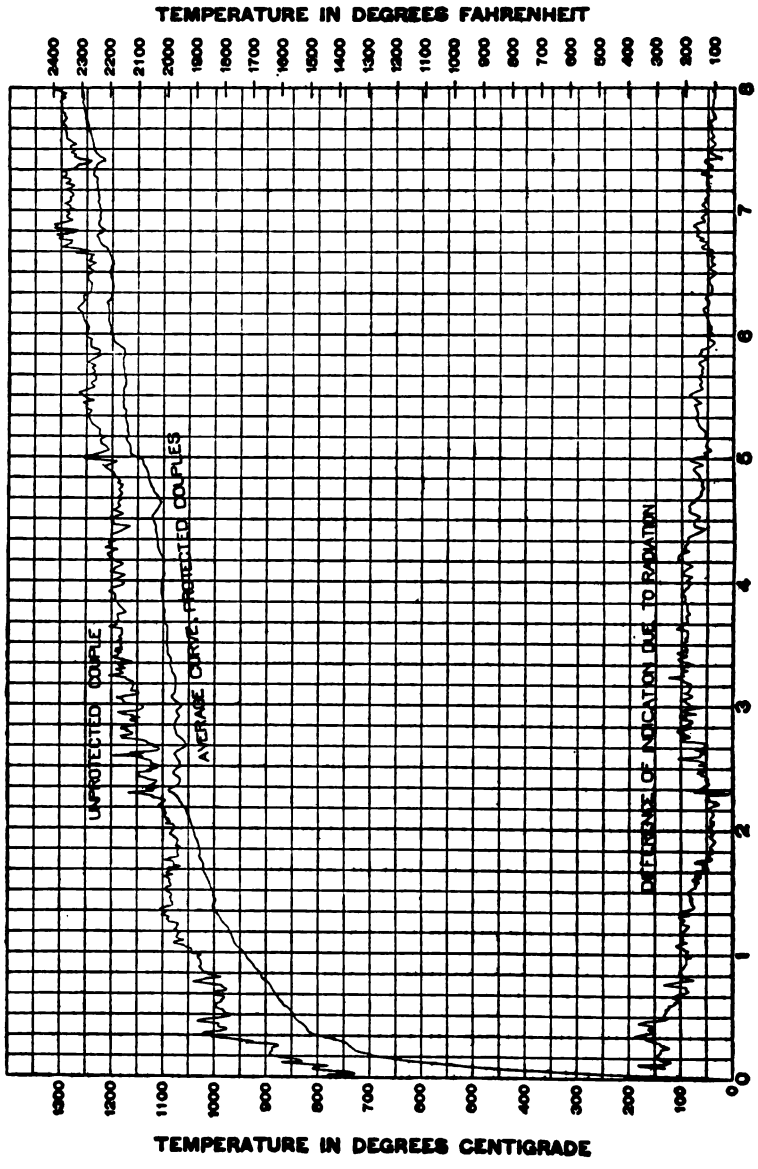

FIG. 41.—*Effect of radiation on indication of furnace pyrometers*

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the decrease is gradual up to the uniform slope after two hours (Fig. 39), where the lag correction is about 1° C. For the furnace temperatures of the individual tests the corrections would be much more irregular than those on the average curve on account of irregularities of slope, although, in general, after the first hour they would be relatively small and within the limit of error applicable to furnace-temperature measurements. The furnace temperatures as given on the plots in Figs. 90 to 145 are indicated temperatures, as it was not deemed practicable to apply corrections for either lag or radiation effects to results of individual tests.

(2) RADIATION EFFECTS.—In a gas-fired furnace where the source of heat is combustion within the chamber the inside of the walls of the latter will be at lower temperature than the furnace contents. The indications of a pyrometer introduced into the chamber to measure the temperature of its contents will be influenced by radiant interchange of heat with the inclosure, the extent of the effect being dependent mainly upon the size of the pyrometer and the temperature difference between the inclosure and the gaseous contents. To obtain a measure of the extent to which the indications of the furnace pyrometers were thus affected, an unprotected thermocouple of fine platinum—platinum-rhodium wires 0.004 inch (1/10 mm) in diameter was supported 1½ inches outside of the closed end of a porcelain tube on similar wires of larger diameter that were passed through and sealed into the end of the tube, through which they led to the outside of the furnace. Two pyrometers of the same design as those used in the column tests (Fig. 32) were placed about 3 inches away from the unprotected couple and several runs made in a furnace having walls of the same material and of about the same thickness as those of the column furnace. Readings of the unprotected and the protected couples were taken at intervals of one-half minute or less. The indicated temperatures obtained in a run extending to 8 hours are plotted in Fig. 41. Those given by the unprotected couple were considerably higher than those of the protected couples, because the temperature of the fine wire was influenced to a much smaller extent by radiant heat interchange with the inside of the furnace inclosure than that of the protected couples, and the temperatures indicated by it approximate those of the furnace contents as closely as can be attained without excessive refinements.

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The difference in indication due to radiation, as given by the lower curve in Fig. 41, was obtained by correcting the average indication of the protected couples for lag and subtracting the corrected value from that of the unprotected couple. While the variations in the resulting difference of indication are partly due to local and general variations in furnace conditions and minor changes in calibration of the unprotected couple, the general decrease from about 150° C during the first half hour to less than 50° C at the end of 8 hours can be ascribed mainly to decrease in temperature difference between the furnace contents and the inside surface of the furnace chamber.

The average difference due to radiation was applied to the results plotted in Fig. 40 (*b*), and the estimated indication of the unprotected fine-wire couple is given by the upper curve.

Since the indication of all pyrometers is influenced by lag and radiation effects, the comparison of furnace exposures afforded by the indicated temperatures given in Fig. 39 is limited by differences in these particulars incident with the types of pyrometers used.

3. FIRE AND WATER TEST PROCEDURE

The duration of the fire periods varied from 22½ minutes to one hour, and that of the subsequent water application from one to five minutes. The length of the maximum fire period was determined by the time within which water is generally applied in building fires, which was estimated as being one hour. The unprotected columns and some of the protected columns were given fire periods of shorter duration which were well within the time to failure of the corresponding columns in the fire-test series. The duration and pressure of the water application were also varied for the different types of protection, the heavier ones being subjected to the most severe test conditions.

In applying the hose stream, the nozzle was moved back and forth on one side of the furnace and maintained at a constant distance from the column, the water being applied in succession over the full height on three of its sides.

4. OBSERVATIONS DURING TEST

Readings for temperature of furnace and of test column were taken at intervals of 2 to 15 minutes, the frequency of the readings

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depending on the rate of temperature change. Readings for deformation were also taken at intervals of 2 to 15 minutes.

Notes were taken on the character of the fire and on its effects on the test column at all stages of the test where the latter could be observed.

5. OBSERVATIONS AFTER FAILURE

After the column had cooled notes were taken of its general condition and the covering was partly or wholly removed to determine the extent to which it was damaged. Measurements were taken of the amount and direction of the final buckle.

6. PHOTOGRAPHIC RECORDS

Photographs were taken of all columns after test and also of a number of typical columns before test. The views were generally taken diagonally and from opposite sides, so as to show all faces of the column, only one view being as a rule included in this paper.

X. RESULTS OF FIRE TESTS

1. FIRE-TEST RESULTS IN TABLES AND FIGURES

The results of the fire tests in points of time to failure, load sustained, and relative fire exposure are given in Tables 42*a* to 42*i*. The columns are grouped by classes of protection, and the thickness and material applied in each test are indicated.

In Table 43 (pp. 122–123) are given the period of expansion, the time to failure, and the maximum temperatures attained in the metal.

The period of expansion of a column when loaded and exposed to fire is the period from the beginning of the test to the time when expansion of the column ceases, due to yielding of the heated metal under the applied load.

The time to failure in the fire test extends from the beginning of the test to the time when the column is unable to sustain the applied working load.

The time to failure of all columns in the fire-test series is given in diagram form in Fig. 42 (p. 114), and the period of expansion of the steel, cast-iron, and concrete columns is given in similar manner in Fig. 45 (p.121).

2. PHOTOGRAPHIC RECORDS

Views before and after test of the columns in the fire-test series are given in Figs. 58 to 82, Appendix A (pp. 216–241).

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TABLE 42a.—RESULTS OF FIRE TESTS

Unprotected columns

Test No.	SECTION	Nominal Area, Sq. In.	l/r	Load Sustained During Test		Time to Failure, Hr—Min.	Furnace Exposure, Percent
				Total Load, Lb.	Unit Load, Lb. per Sq. In.		
1	Rolled H	10.17	75.6	119500	11750	0—11¼	95.6
2	Plate and Angle	18.00	111.8	116000	8900	0—19¾	97.0
3	Plate and Channel	8.76	64.7	111000	12650	0—14	101.4
4	Latticed Channel	7.78	44.0	111000	14250	0—11	101.1
5	Z-bar and Plate	9.32	81.7	105000	11250	0—14¼	100.0
6	I-beam and Channel	10.12	72.1	122000	12050	0—17	89.6
7	Latticed Angle	8.44	40.7	122500	14500	0—14	96.3
8	Starred Angle	13.27	108.5	124000	9350	0—21½	99.0
9	Round Cast Iron	14.73	68.2	95500	6500	0—34¼	101.2
10	Round Cast Iron	14.73	68.2	95500	6500	0—34½	101.7
10A	Round Cast Iron	14.45	63.2	98500	6800	0—34¼	100.3
11	Round Cast Iron(Concrete filled)	14.73	68.2	95500	.....	0—45¼	103.1
12	Steel Pipe (Concrete filled)	Steel 6.93 Concrete 38.74	63.9	114500	.....	0—36	101.9
3	Reinforced Steel Pipe (Starred angles imbedded in concrete filling)	Steel 18.36 Concrete 40.07	68.2	236000	.....	1—11¼	99.8

NOTE: Details of the steel, cast-iron, and pipe columns listed in Tables 42*a* to 42*g* are given in Figs. 1 to 4 (p. 16–19), and Figs. 6 and 7 (p. 22 and 24). Further properties of the sections are given in Tables 3*a* to 3*h* (p. 33–47).

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TABLE 42b.—RESULTS OF FIRE TESTS

Columns partly protected by concrete

Test No.	SECTION	*Protection		Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
		Mixture	Kind of Concrete				
14	Rolled H.....	1:2:4	Joliet gravel.....	405	119500	1 — 04¼	85.1
15	Rolled H.....	1:2:4	Rockport granite....	407	119500	0 — 48¾	91.5
16	Plate and Angle...	1:2:4	New York trap.....	416	116000	0 — 44½	94.9
17	Plate and Angle...	1:1½:4½	Hard coal cinders...	408	116000	0 — 41¾	99.6
18	Latticed Channel..	1:2:4	New York trap.....	418	111000	2 — 53	98.1
19	Z-bar and Plate....	1:3:5	Chicago limestone...	414	105000	1 — 07¾	95.8
20	I-beam and Channel.....	1:3:5	New York trap.....	415	122000	1 — 24½	100.2
21	I-beam and Channel.....	1:3:5	New York trap.....	416	122000	1 — 21¾	99.6
22	Latticed Angle....	1:2:4	Chicago limestone...	408	119500	5 — 14	99.2

*Re-entrant portions and interior filled with concrete.

TABLE 42c.—RESULTS OF FIRE TESTS

Columns protected by plaster on metal lath

Test No.	SECTION	Protection	Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
23	Plate and Angle...	Two 2-coat layers of Portland cement plaster on expanded metal lath, each layer 1 in. thick, with a ¾-in. air space between layers	508	*117500	2 — 52	103.1
24	Plate and Channel.	Two 2-coat layers of Portland cement plaster on woven wire lath, each layer ⅞ in. thick with a ¾-in. air space between layers	496	111000	2 — 24	101.5
25	Z-bar and Plate...	One 2-coat layer of Portland cement plaster, 1 in. thick, on expanded metal lath	484	105000	1 — 07¾	103.7
26	Latticed Angle....	One 2-coat layer of Portland cement plaster, 1½ in. thick, on expanded metal lath	497	122500	1 — 23½	104.2
27	Round Cast Iron..	One 2-coat layer of Portland cement plaster, 1½ in. thick on high ribbed expanded metal lath with a ½-in. broken air space	498	96500	2 — 58	98.2

*Heavier load used as plate has 1/32 in. greater thickness than nominal.

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TABLE 42d.—RESULTS OF FIRE TESTS

Columns protected by concrete

Test No.	SECTION	Protection			Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
		Thickness, In.	Mixture	Kind of Concrete				
28	Rolled H	2	1:2:4	Chicago limestone	437	119500	6 — 33½	98.9
28A	Rolled H	2	1:2:4	Chicago limestone	438	119500	7 — 09½	98.2
29	Rolled H	2	1:2:4	New York trap	435	119500	4 — 35½	98.5
30	Rolled H	2	1:2:4	Joliet gravel	439	119500	7 — 16	98.2
31	Rolled H	2	1:2:4	Cleveland sandstone	509	119500	4 — 11½	98.4
32	Rolled H	2	1:2:5	Hard coal cinders	509	119500	3 — 44	101.3
32A	Z-bar and Plate	2	1:2:5	Hard coal cinders	497	105000	4 — 02	100.5
33	Rolled H	4	1:2:4	Chicago limestone	450	119500 ‡433000	8 — 08	98.2
33A	Rolled H	4	1:2:4	Chicago limestone	455	119500 ‡406000	8 — 07½	98.8
34	Rolled H	4	1:2:4	Rockport granite	452	119500	7 — 58	98.6
34A	Rolled H	4	1:2:4	Rockport granite	454	119500	7 — 23	100.6
35	Rolled H	4	1:3:5	Chicago limestone	504	119500 ‡348000	8 — 07	101.3
36	Plate and Angle	2	1:2:4	New York trap	445	116000	3 — 53½	100.3
37	Plate and Angle	4 round	1:2:4	New York trap	503	116000	7 — 34½	98.9
38	Plate and Channel	2	1:2:4	Joliet gravel	449	*116500	5 — 28½	100.2
39	Plate and Channel	4	1:2:4	Meramec R. gravel	436	*116500	3 — 41½	98.3
40	Latticed Channel	2 round	1:2:4	New York trap	501	111000	7 — 57	99.1
41	Z-bar and Plate	4	1:2:5	Chicago limestone	451	105000 ‡333000	8 — 24½	98.6
42	Z-bar and Plate	4	1:2:5	Chicago limestone	453	105000 ‡333000	8 — 11½	98.5
43	I-beam and Channel	2	1:2:4	Cleveland sandstone	456	122000	4 — 11	98.7
44	I-beam and Channel	2	1:3:5	Cleveland sandstone	458	122000	3 — 04½	100.6
45	Starred Angle	2 round	1:2:4	Meramec R. gravel	447	124000	1 — 47	98.7
46	Latticed Angle	†2	1:2:4	New York trap	451	122500	6 — 42½	99.0
47	Round Cast Iron	2	1:2:5	Hard coal cinders	446	95500	2 — 48½	99.6

* Heavier load used as plates have 1/32 in. greater thickness than nominal.
† 2-in. outside rivets, 3½-in. outside angles.
‡ Load necessary to cause failure of column. After 8 hr. the load was increased until failure occurred.

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TABLE 42e.—RESULTS OF FIRE TESTS

Columns protected by hollow clay tile

Test No.	SECTION	Protection		Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure Percent
		Thickness of Tile, In.	Kind of Tile, Filling, and Method of Tying				
48	Rolled H.....	2	New Jersey semi-fire clay..... No filling Outside wire ties	496	119500	1 — 50	100.9
49	Rolled H.....	4	Same as No. 48.....	497	119500	1 — 40	99.9
50	Plate and Angle...	2	Surface clay, Boston.. Granite concrete fill No ties	494	116000	1 — 05¾	99.8
50A	Plate and Angle...	2	Same as No. 50.....	505	*117500	1 — 59½	101.8
51	Plate and Angle...	4	Surface clay, Boston Granite concrete fill Outside wire ties	487	116000	2 — 17¾	101.8
51A	Plate and Angle...	4	Same as No. 51.....	507	116000	2 — 55½	100.5
52	Plate and Channel.	2	Ohio shale..... Cinder concrete fill Outside wire ties	513	111000	1 — 40¾	100.2
53	Plate and Channel.	4	Same as No. 52.....	495	111000	1 — 22¾	103.0
54	Latticed Channel..	2	Ohio semi-fire clay.... Trap concrete fill Outside wire ties	489	111000	3 — 17¾	101.1
55	Z-bar and Plate....	2	Ohio semi-fire clay.... Limestone concrete fill Outside wire ties	485	105000	3 — 46¾	99.5
56	Z-bar and Plate....	4	Ohio semi-fire clay.... Limestone concrete fill Wire mesh in joints	491	105000	3 — 33½	100.3
57	I-beam and Channel.....	4	Surface clay, Chicago.. Limestone concrete fill Outside wire ties	491	122000	3 — 23	100.0
58	I-beam and Channel.....	Two 2	Surface clay, Chicago.. Hollow tile fill Wire mesh in joints	502	122000	4 — 35¾	101.7
59	I-beam and Channel.....	Two 2	Surface clay, Chicago.. Hollow tile fill Outside wire ties	490	122000	1 — 33¾	101.2
60	Latticed Angle....	2	Ohio semi-fire clay.... Trap concrete fill, placed before tile was set Outside wire ties	487	122500	3 — 09½	100.4
61	Latticed Angle....	2	Ohio semi-fire clay.... No filling Outside wire ties	501	122500	0 — 50¾	100.3
62	Round Cast Iron..	2 round	Porous semi-fire clay, New Jersey..... No filling Outside wire ties	483	95500	4 — 11½	101.3
63	Round Cast Iron..	2 round	Same as No. 62.....	493	95500	2 — 57½	100.6
76	Rolled H.....	2	Ohio shale; Ohio semi-fire clay; semi-fire clay, New Jersey.... Limestone concrete fill Wire mesh in joints Tile covered with ¾-in. layer of gypsum plaster	42	119500	4 — 25½	98.1
77	Plate and Angle	4	Semi-fire clay, N. J.; surface clay, Chicago; surface clay, Boston. Limestone concrete fill Wire mesh in joints Tile covered with ¾-in. layer of lime plaster	45	116000	4 — 42¾	97.3

* Heavier load used as plate has 1/32 in. greater thickness than nominal.

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TABLE 42f.—RESULTS OF FIRE TESTS

Columns protected by gypsum block

Test No.	SECTION	Thickness of Block, In.	Protection: Kind of Block, Filling and Method of Tying	Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
64	Rolled H..	4	Western gypsum (solid) Hollow gypsum block fill Wall ties in joints	502	119500	4 — 43½	104.5
65	Plate and Channel	2	Western gypsum (solid) Solid gypsum block fill Wall ties in joints	505	111000	2 — 21½	104.5
66	Latticed Channel	2	Eastern gypsum (solid) Poured gypsum fill Wire mesh in joints	496	111000	2 — 36	101.0
67	Rolled H..	4	Same as No. 66	492	119500	5 — 31½	101.2
67A	Rolled H..	4	Same as No. 66	491	119500	6 — 24½	99.7

TABLE 42g.—RESULTS OF FIRE TESTS

Columns protected by brick

Test No.	SECTION	Thickness of Brick, In.	Kind of Brick and Filling	Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
68	Rolled H..	2¼	Chicago common brick set on edge and end Brick fill	498	119500	1 — 40¾	104.0
69	Rolled H..	3¾	Chicago common brick laid flat Brick fill	502	119500	7 — 13½	101.2

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TABLE 42h.—RESULTS OF FIRE TESTS

Reinforced concrete columns

Effective length, 12 ft., 8 in.

Test No.	SECTION	Outside Dimensions and Reinforcement	Concrete		Age, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
			Mixture	Kind.				
70	Square Vertically Reinforced	16-in. square..... Four 1-in. sq. bars	1:2:4	Chicago limestone	433	101000 ‡294000	8 — 40¼	101.1
71	Square Vertically Reinforced	Same as No. 70....	1:2:4	New York trap	450	101000	7 — 22¾	99.2
72	Round Vertically Reinforced	17-in. diameter.... Six 1-in. sq. bars	1:2:4	Chicago limestone	520	107500 ‡250000	8 — 04½	102.1
73	Round Vertically Reinforced	Same as No. 72....	1:2:4	New York trap	442	107500	7 — 57½	98.5
74	Hooped Reinforced	17-in. diameter.... Six ¾-in. sq. bars ¼-in ϕ spiral on 1½-in pitch	1:2:4	Chicago limestone	522	129000 ‡243000	8 — 06½	99.5
75	Hooped Reinforced	Same as No. 74....	1:2:4	New York trap	460	129000 ‡163000	8 — 01¼	100.7

‡Load necessary to cause failure of column. After 8 hr. the load was increased until failure occurred.

TABLE 42i.—RESULTS OF FIRE TESTS

Timber columns

Nominal net section, 11⅜ by 11⅜ in. Effective length, 12 ft., 8½ in.

Test No.	Species	Bearing Details	Protection of Column and Cap	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
78	Longleaf pine.	Cast iron cap and pintle	One 2-coat layer of Portland cement plaster 1 in. thick on woven wire lath, ¾-in. air space. Age, 30 days	118500	2 — 15¼	97.6
79	Longleaf pine.	Cast iron cap and pintle	Unprotected.....	118500	0 — 50	105.2
80	Longleaf pine.	Steel plate cap and timber strut	One thickness of ⅜-in. gypsum wall board with metal corner beads nailed to column	118500	1 — 13	103.5
81	Longleaf pine.	Steel plate cap and timber strut	Unprotected.....	118500	0 — 35	113.0
82	Douglas fir....	Cast iron cap and pintle	Unprotected.....	118500	0 — 45¼	106.6
83	Douglas fir....	Steel plate cap and timber strut	Unprotected.....	118500	0 — 38½	106.1

NOTE: Details of design of reinforced-concrete and timber columns are given in Figs. 8 to 10 (p. 26–29), and further details of protection are given in Tables 3*i* and 3*j* (p. 48–49).

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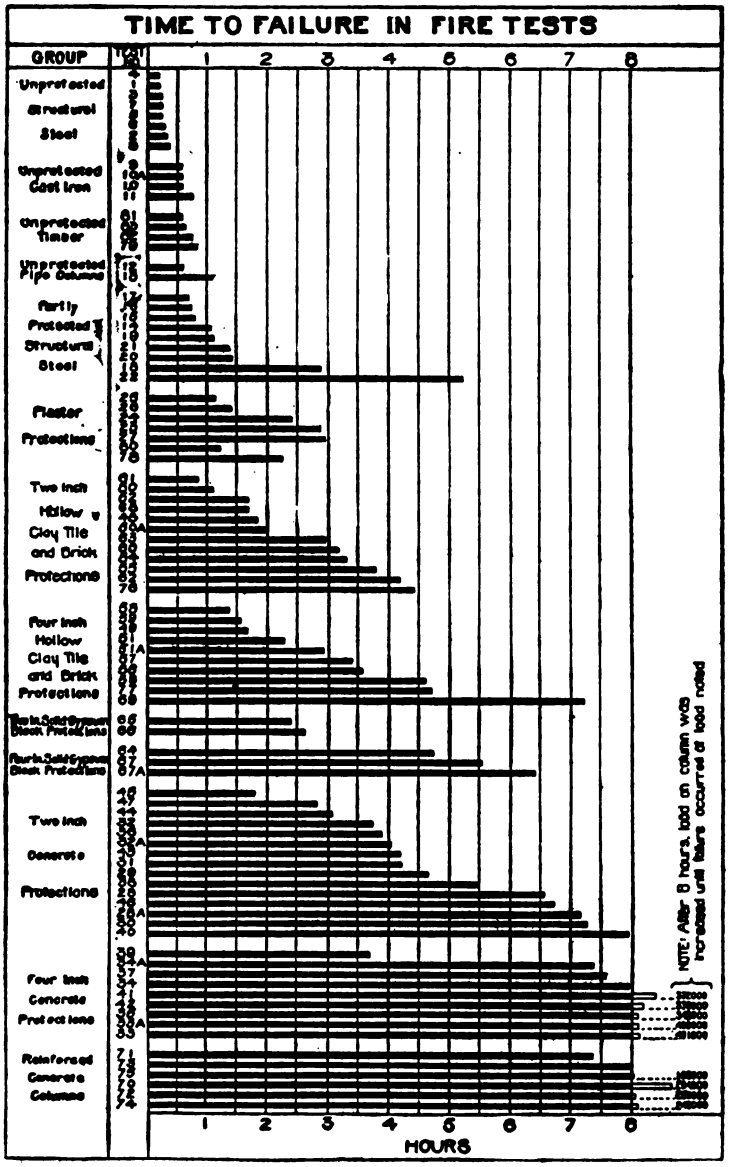
TIME TO FAILURE IN FIRE TESTS
GROUP
TEST NO.
1 2 3 4 5 6 7 8
Unprotected Structural Steel
Unprotected Cast Iron
Unprotected Timber
Unprotected Pipe Columns
Partly Protected Structural Steel
Plaster Protections
Two Inch Hollow Clay Tile and Brick Protections
Four Inch Hollow Clay Tile and Brick Protections
Two Inch Concrete Protections
Four Inch Concrete Protections
Reinforced Concrete Columns
NOTE: After 8 hours, load on column was increased until failure occurred or load noted
HOURS

FIG. 42.—*Time to failure of columns in fire test series*

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Since a large portion of the effects shown on the photographs of columns after test is due to the deformation and deflection taking place at failure, they should not be considered as representing the condition of the column or its covering immediately before this point was reached. The general condition of the test columns near the end of the fire period is indicated in their respective test logs.

3. FURNACE TEMPERATURES

The temperatures of the furnace in the fire tests, as indicated by thermocouples located on two levels at symmetrical points within the chamber, are given by the upper curves in Figs. 90 to 141, Appendix B (pp. 249–302).

(*a*) VARIATIONS FROM AVERAGE CURVE

To obtain a measure of the variation in furnace exposure between the different tests, the area under the average furnace curve given in Fig. 39, was calculated up to the end of successive intervals and compared with the area under the average furnace curve for each test up to a point near failure. The comparisons are given in Tables 42*a* to 42*i* as percentages of the area under the average curve of all tests.

4. COLUMN TEMPERATURES

The temperatures attained in the test columns and their coverings are given by the lower curves in Figs. 90 to 141, Appendix B. The arrows on the plots indicate the time of failure for each test.

(*a*) TEMPERATURE VARIATION OVER LENGTH OF COLUMN

The highest temperature generally obtained over the middle half of the column, the loss of heat at the ends due to conduction tending to maintain a lower temperature in the adjacent portions of the column. The difference was seldom very large, and in some tests, due to local variations in furnace temperature or in column protection, the highest temperature was indicated by one of the end couples.

(*b*) TEMPERATURE VARIATION OVER CROSS SECTION

The temperature variation laterally across the section was quite marked in all protected-column tests, the highest temperature in the metal obtaining almost invariably at the outer edges. In Fig. 43 this variation is given for some typical columns near failure.

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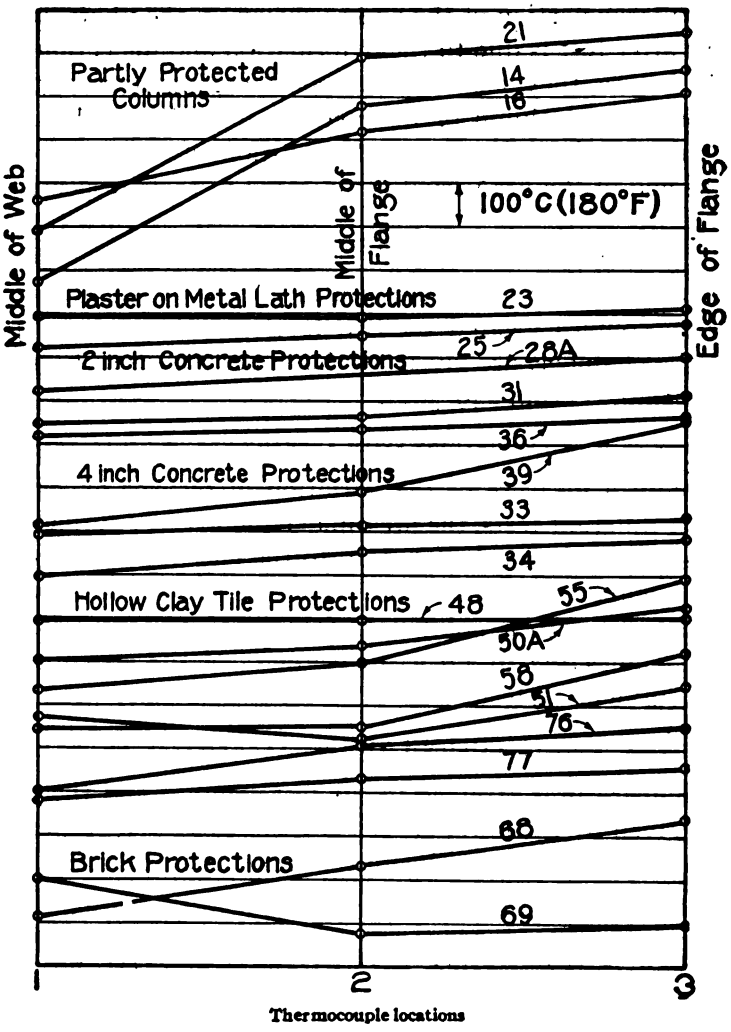

FIG. 43.—*Temperature variation over cross-section of typical columns*

The temperature indicated by the couple in the covering varied with its distance from the surface and the character of the covering material.

(*c*) DEHYDRATION POINTS

In the tests with concrete or gypsum protections and reinforced-concrete columns, temperatures in the column of about 100° C obtained over a considerable length of time, due to evaporation of water in the concrete and gypsum. To less extent this effect was present in tests with plaster on metal-lath and concrete-filled hollow clay-tile protections.

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5. LONGITUDINAL DEFORMATION AND AVERAGE TEMPERATURE

The unit longitudinal deformation measured over a 37-inch gage length located immediately above the midheight of the column (Fig. 33), together with the computed average effective temperature over the same length, are plotted for a number of fire tests in Figs. 146 to 171, Appendix C (pp. 307–333).

(a) COMPUTATION OF AVERAGE EFFECTIVE TEMPERATURE

The relative location of gage points and thermocouple points and the temperature variation between the latter assumed in calculating the average effective temperature in the gage length, are shown in Fig. 44 for the case of the rolled H section. It can

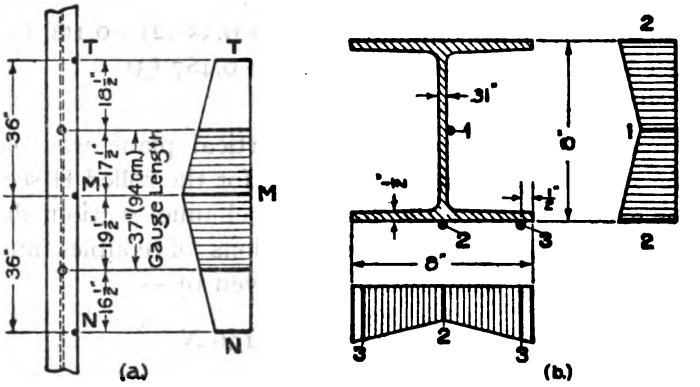

FIG. 44.—*Assumed temperature variation between thermocouple points*

be readily shown with reference to the diagram at (*a*) that the average temperature in the gage length of the edge couples, N, M, and T, located in the (3) position, is given by

$$0.115\ T + 0.143\ N + 0.742\ M = \text{Average (3)},$$

where T, N, and M are the temperatures indicated by the respective thermocouples at a given time. Similarly with reference to the diagram at (*b*) it can be shown that the average temperature in the cross section at a given level, say at the M position, is expressed by,

$$0.17\ (1) + 0.38\ (2) + 0.45\ (3) = \text{Average } M,$$

(1), (2), and (3) being the temperatures indicated by thermocouples in the respective positions in the cross section. The average effective temperature in the gage length (Av. *G L*) can then be obtained from the relation,

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$$\text{Average } G\,L = \left(\frac{\text{Average } 3}{M\ 3}\right) \text{Average } M.$$

For the other structural-steel sections the average temperature at the *M* or *N* level, was obtained by taking the sum of parts of temperature readings of couples in the (1), (2), and (3) positions as given below, the derivation being similar to that for the rolled H section:

Plate and angle 0.33 (1) +0.38 (2) +0.29 (3)
Plate and channel 0.20 (1) +0.27 (2) +0.53 (3)
Latticed channel 0.27 (1) +0.365 (2) +0.365 (3)
Z bar and plate 0.441 (1) +0.232 (2) +0.327 (3)
I beam and channel 0.223 (1) +0.294 (2) +0.483 (3)
Latticed angle, test No. 26 0.484 (1) +0.35 (2) +0.166 (3)
Latticed angle, test No. 60 0.513 (2) +0.487 (3)
Starred angle, 0.416 (1) +0.584 (2)

The average temperature in the vertical plane was obtained from the same expression as that given for the rolled H section in all cases except that for the plate and channel section, due to a slight difference in the relative positions of couples and gage points, the average temperature was given by—

$$0.122\,T + 0.742\,M + 0.136\,N.$$

In the case of the cast-iron and pipe columns the average temperatures plotted pertain to the outer surface of the metal. Readings obtained on the inside of a number of the unfilled cast-iron columns indicate temperatures generally lower by 5 to 10° C. For the reinforced-concrete columns, Figs. 169 to 171, the temperatures plotted are those of the vertical reinforcing bars.

The maximum error involved in computing average temperatures by the above method is estimated to be within 20° C for tests where the covering material remained in place until near the time of failure. Where parts of the covering fell off, as for tests plotted in Figs. 163, 165, and 166, the resulting local heating and irregular temperature distribution introduced much larger errors, as evidenced by the discrepancy between the computed temperature and the corresponding unit deformation. For such tests the limit of error is probably as high as 50° C.

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(*b*) DEFORMATION UNDER HEAT AND LOAD

On application of working load to the column before starting the fire test, a compressive deformation resulted, which in a few tests increased slightly during the first part of the fire period, caused either by shrinkage stresses set up by dehydration of the covering or by increased load on the metal portion of the column resulting from cracking of the covering. As the temperature increased the steel, cast-iron, and reinforced-concrete columns expanded up to a point where the yielding due to the load became equal to or greater than the thermal expansion. (See curves, pp. 308–333).

The unit expansion per degree C during this period varied generally between 0.000010 and 0.000014 for the steel and cast-iron columns, the average of the calculated values for each of these two types of columns being very nearly 0.0000125. These were taken from room temperature up to a point where the rate of expansion began to decrease rapidly due to yielding of the metal. The total observed unit expansion up to the point where expansion ceased varied in the tests of steel columns from 0.0044 to 0.0066, the average being 0.0054. For the cast-iron columns the variation was from 0.0060 to 0.0071, with an average of 0.0064. The lower values are due mainly to local heating which caused the metal to yield over a relatively short portion of the gage length before full expansion had been attained elsewhere. The columns under the lower unit loads generally attained higher expansion than those more heavily loaded.

In the tests of reinforced-concrete columns unit expansions of 0.0000095 to 0.0000102 per degree C obtained, with values of maximum unit expansions of 0.0023 to 0.0046. The point of maximum expansion was less sharply defined than for the steel and cast-iron columns, and the expansion attained was lower.

The compressive deformation taking place subsequent to maximum expansion varied in rate and duration with the type of column and the load-carrying capacity of the covering material. In some tests the total compression before failure more than equaled the previous expansion.

In the tests of timber columns slight expansions were noted during the first few minutes of the fire period, subsequent to which the movement was one of progressive depression, the principal deformation occurring at the bearings on the steel or cast-iron cap introduced near the top of the column.

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(*c*) PERIOD OF EXPANSION

The period of expansion and time to failure of all columns in the fire-test series are given in Table 43 (pp. 122–123). A comparison is given between the length of the portions of the test period preceding and following the point of maximum expansion, as also the maximum temperatures obtaining at the latter point and at failure.

The period of expansion of the steel, cast-iron, and concrete columns is shown in chart form in Fig. 45.

(*d*) MAXIMUM COLUMN TEMPERATURES

These are given in Table 43 for the point of maximum expansion and the time of failure, the temperatures being obtained from the time-temperature curves extended where necessary to the failure point in the given test.

The edge temperature for the structural-steel columns is the maximum temperature indicated by the couple located nearest to the edge of the section on any of the four regular thermocouple levels. For the cast-iron columns the temperatures given pertain to the outer surface of the metal and for the reinforced-concrete columns to the vertical reinforcing bars. The edge temperatures given for the timber columns were measured on the metal cap at the edge of the column bearing.

The maximum average temperatures for the sections of the columns having the highest edge temperatures were computed by the method given in paragraph 5*a* above, the limits of error involved being somewhat higher than for the plotted results, as each determination is based on fewer couple readings. In the case of the unprotected and partly protected columns, gypsum-block protections, and a few of the concrete and hollow clay-tile protections the average temperatures are not given, since the rapid temperature rise and irregular distribution obtaining near the end of these tests made the computed results unreliable.

At the point of maximum expansion the average over the sections having the maximum temperature varies for tests of structural steel from 484 to 593° C (903 to 1099° F), with an average of 530° C (986° F). Similarly at the time of failure in the fire tests the computed values vary for the given structural-steel columns from 570 to 837° C (1058 to 1601° F), the average being 668° C (1234° F). The high temperatures obtaining at failure in a number of the tests indicate that at this point the covering carried

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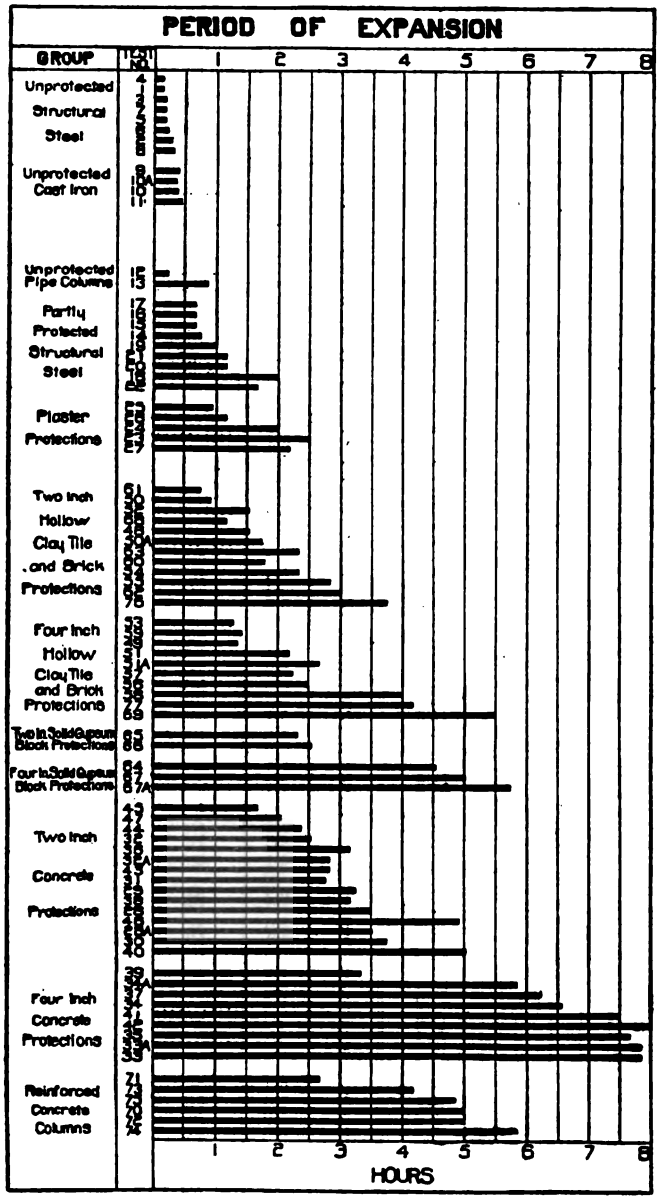
PERIOD OF EXPANSION
GROUP
1 2 3 4 5 6 7 8
Unprotected Structural Steel
Unprotected Cast Iron
Unprotected Pipe Columns
Partly Protected Structural Steel
Plaster Protections
Two Inch Hollow Clay Tile and Brick Protections
Four Inch Hollow Clay Tile and Brick Protections
Two in Solid Gypsum Block Protections
Four in Solid Gypsum Block Protections
Two Inch Concrete Protections
Four Inch Concrete Protections
Reinforced Concrete Columns
HOURS

FIG. 45.—*Expansion period of steel, cast iron, and concrete columns in fire-test series*

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TABLE 43.—TIME TO FAILURE, PERIOD OF EXPANSION AND MAXIMUM COLUMN TEMPERATURES

Test No.	Period of Expansion Hr.—Min.	Time to Failure, Hr.—Min.	Difference, Per cent of Expansion Period	Maximum Observed Temperature in Metal, Deg. C.					
				At End of Expansion Period			At Failure		
				Couple Level	Edge	Average	Couple Level	Edge	Average
1	0—08	0—11¼	40.6	B	530	.....	B	624	.....
2	0—17	0—19¼	13.2	N	633	.....	N	668	.....
3	0—11	0—14	27.3	M	557	.....	M	626	.....
4	0—09	0—11	22.2	M	510	.....	M	578	.....
5	0—11	0—14¼	29.5	M	620	.....	M	670	.....
6	0—13	0—17	30.8	B	623	.....	B	660	.....
7	0—11	0—14	27.3	M	547	.....	M	622	.....
8	0—19	0—21½	13.1	B	575	.....	B	620	.....
9	0—24	0—34¼	42.7	M	584		M	694	
10	0—23	0—34½	50.0	B	638	Measured on surface of column	B	745	Measured on surface of column
10A	0—22	0—34¼	55.7	N	626		N	718	
11	0—27	0—45¼	67.6	B	570		B	758	
12	0—14	0—36	157.0	N	502		.....	.....	
13	0—52	1—13¼	37.9	M	781		M	872	
14	0—45	1—04¼	42.8	M	623	.....	M	810	.....
15	0—40	0—48¼	20.6	N	670	.....	N	730	.....
16	0—40	0—44½	11.2	M	671	.....	M	717	.....
17	0—40	0—41¾	4.4	N	700	.....	N	724	.....
18	2—00	2—53	44.1	N	672	.....	N	788	.....
19	1—00	1—07¼	12.1	M	831	.....	M	860	.....
20	1—10	1—24½	20.7	N	853	.....	N	909	.....
21	1—10	1—21¾	16.8	M	820	.....	M	858	.....
22	1—40	5—14	214.0	M	504	.....	N	790	.....
23	2—30	2—52	14.7	M	573	556	M	650	635
24	2—00	2—24	20.0	N	504	498	N	614	609
25	0—57	1—07¾	18.9	N	564	522	N	653	610
26	1—10	1—23½	19.3	N	512	501	N	606	598
27	2—11	2—58	35.9	T	607	.....	N	735	.....
28	3—30	6—33¾	87.5	M	590	545	N	750	716
28A	3—31	7—09¼	103.5	N	515	484	N	723	690
29	3—15	4—38½	42.8	N	565	525	N	735	693
30	3—45	7—16	93.8	N	570	535	.....	.....	.....
31	2—46	4—11½	51.5	N	533	504	N	732	702
32	2—32	3—44	47.4	N	547	503	B	722	685
32A	2—50	4—02	42.4	M	576	496	N	736	656
33	7—50	*8—08	.....	B	546	533	B	*554	*541
33A	7—50	*8—07¼	.....	M	536	522	M	*547	*532
34	6—34	7—58	21.3	N	598	574	N	718	690
34A	5—50	7—23	26.6	N	571	542	N	732	700
35	7—40	*8—07	.....	N	539	527	N	*558	*546
36	3—10	3—53¼	22.8	B	603	571	B	695	664
37	6—10	7—34½	22.8	B	545	525	B	664	644
38	3—10	5—28¾	73.0	N	531	516	N	696	683
39	3—20	3—41¼	10.6	M	759	.....	M	794	.....
40	5—01	7—57	58.5	B	574	536	M	881	837
41	7—30	*3—24¼	.....	N	527	496	N	*561	*535
42	8—00	*3—11½	.....	B	553	526	B	*560	*535
43	2—50	4—11	47.6	N	601	553	N	769	724
44	2—23	3—04½	28.8	N	562	520	N	727	677
45	1—41	1—47	5.9	B	519	510	B	575	570
46	4—55	6—43¼	36.8	N	543	.....	N	695	.....
47	2—03	2—48¾	37.2	N	568	.....	N	710	.....
48	1—32	1—50	19.6	M	542	537	M	647	636
49	1—21	1—40	23.4	M	561	.....	M	654	.....
50	0—55	1—05¾	19.5	N	682	.....	N	775	.....
50A	1—44	1—59½	14.9	B	695	.....	B	778	.....
51	2—11	2—17¼	4.8	N	617	546	N	670	601
51A	2—40	2—55½	9.7	N	600	549	N	678	622
52	1—32	1—40¾	9.5	N	520	497	N	593	570
53	1—17	1—22¼	6.8	N	580	535	N	659	603
54	2—20	3—17¼	40.9	B	543	493	B	820	764
55	2—51	3—46¾	32.6	M	684	527	M	889	692
56	2—30	3—33½	42.3	M	643	564	M	795	732
57	2—15	3—23	50.4	N	749	593	T	894	755
58	4—00	4—35¾	14.9	T	624	523	T	714	591
59	1—25	1—33¾	10.3	.....	.....	.....	.....	.....	.....
60	1—47	3—09½	77.0	N	561	532	N	750	737
61	0—45	0—50¼	11.7	N	585	571	N	638	625
62	3—00	4—11½	39.7	M	584	.....	M	760	.....
63	2—20	2—57½	26.8	N	598	.....	N	730	.....

*Column loaded to failure after 8-hr. fire exposure.

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TABLE 43.—TIME TO FAILURE, PERIOD OF EXPANSION, AND MAXIMUM COLUMN TEMPERATURES—Concluded

Test No.	Period of Expansion Hr.—Min.	Time to Failure, Hr.—Min.	Difference, Percent of Expansion Period	Maximum Observed Temperature in Metal, Deg. C.					
				At End of Expansion Period			At Failure		
				Couple Level	Edge	Average	Couple Level	Edge	Average
76	3—45	4—25½	18.0	N	553	524	N	663	619
77	4—10	4—42¼	12.9	N	600	558	B	690	652
64	4—32	4—43¼	4.1	T	513	.....	T	868	.....
65	2—20	2—21½	1.1	N	479	.....	N	490	.....
66	2—32	2—36	2.6	T	463	.....	T	491	.....
67	5—01	5—31½	10.1	N	496	.....	.....	.....	.....
67A	5—45	6—24½	11.4	.....	.....	.....	M	980	.....
68	1—10	1—40¾	43.9	N	606	516	N	754	677
69	5—30	7—13¼	31.3	N	550	553	N	706	716
70	5—00	*8—40¼	.....	N	642	.....	N	*797	.....
71	2—40	7—22¾	176.6	T	468	Measured	B	942	Measured
72	5—00	*8—04½	.....	T	475	on vertical	T	*823	on vertical
73	4—10	7—57½	91.0	N	528	reinforcing	N	845	reinforcing
74	5—50	*8—06½	.....	B	493	bars	N	*589	bars
75	4—50	*8—01¾	.....	B	552	.....	B	*788	.....
78	.....	2—15¼	.....	.....	.....	.....	H	360	.....
79	.....	0—50	.....	.....	.....	.....	H	510	.....
80	.....	1—13	.....	.....	.....	.....	H	564	.....
81	.....	0—35	.....	.....	.....	.....	H	432	.....
82	.....	0—45¼	.....	.....	.....	.....	H	510	.....
83	.....	0—38½	.....	.....	.....	.....	H	544	.....

* Column loaded to failure after 8-hr. fire exposure. H. temperature measured on metal cap at edge of timber column bearing.

a large proportion of the applied load. The same effect appears to have influenced to much smaller extent the temperatures attained at the point of maximum expansion.

The surface temperatures at failure and at maximum expansion in the tests of cast-iron columns were on the average about 70° C (126° F) higher than the average over the section for structural-steel columns at the given stages of the test.

6. TOTAL VERTICAL DEFORMATION

(*a*) BEFORE FAILURE

Measurement of the movement of the head of the loading ram was made for a number of columns as described in Sec. VIII, paragraph 4, and the results are plotted in Figs. 46 and 47. An approximate measure of the total expansion and depression was also obtained from the card of the indicator (Sec. VI, par. 3*b*) mounted on the control board.

The total expansion of the steel columns varied from ⅞ to ⅛ inch and of the cast-iron columns from 1 to ½ inch, the lower values being due to local heating caused by impairment of the covering over a short length, which induced failure while other

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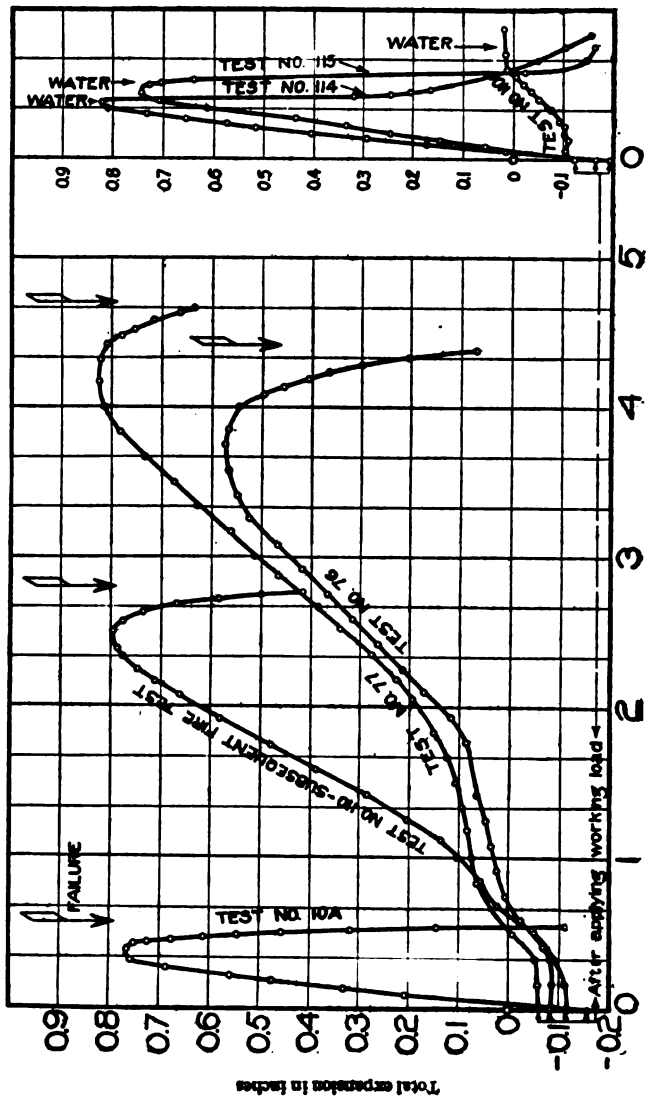

FIG. 46.—*Total expansion, test Nos. 10A, 76, 77, 110, 114, and 115*

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portions of the column were at much lower temperature. The expansion of the reinforced-concrete columns was about one-half that of the cast-iron columns.

The depression of the top of the timber columns is given in Fig. 47. These deformations were due mainly to crushing of the wood at the metal cap, the heat conducted by it into the bearing causing a large reduction in the compressive strength of the wood in contact with it.

(*b*) AT FAILURE

With the setting used for the cut-off valve of the cylinder, the resulting downward movement at failure varied between 2½ and 3¼ inches. As given on the indicator card, from ¼ to 2½ inches of this travel was made under nearly full pressure. The indicated pressure in the cylinder at the end of the travel varied from ¼ to ½ of the original pressure in tests with steel columns and some of the cast-iron columns. In the case of the cast-iron columns that broke before the end of the travel and of the reinforced-concrete columns that failed under working load, the pressure indicated immediately before the valve cut off was less than ¼ of the original pressure.

7. LATERAL DEFLECTION

(*a*) BEFORE FAILURE

The center deflections observed, where large enough to have any bearing on the manner of failure of the column, are noted in the respective test logs. In most tests, decided deflection did not begin until after the point of maximum expansion was passed. The deflection observed at the last reading before failure, varied for different tests from less than ⅛ to 2½ inches. In almost all cases the direction of the deflection before failure was the same as that of the final buckle. Initial general bends up to $\frac{3}{16}$ of an inch appear to have had no influence on the direction of the deflection of the column either before or at failure.

The maximum measured deflections of the reinforced-concrete columns before the end of the 8-hour fire period varied from less than ⅛ to ¼ of an inch.

(*b*) DEFLECTION AT FAILURE

The maximum deflections at failure of the steel columns as measured after they were taken out of the furnace, varied from

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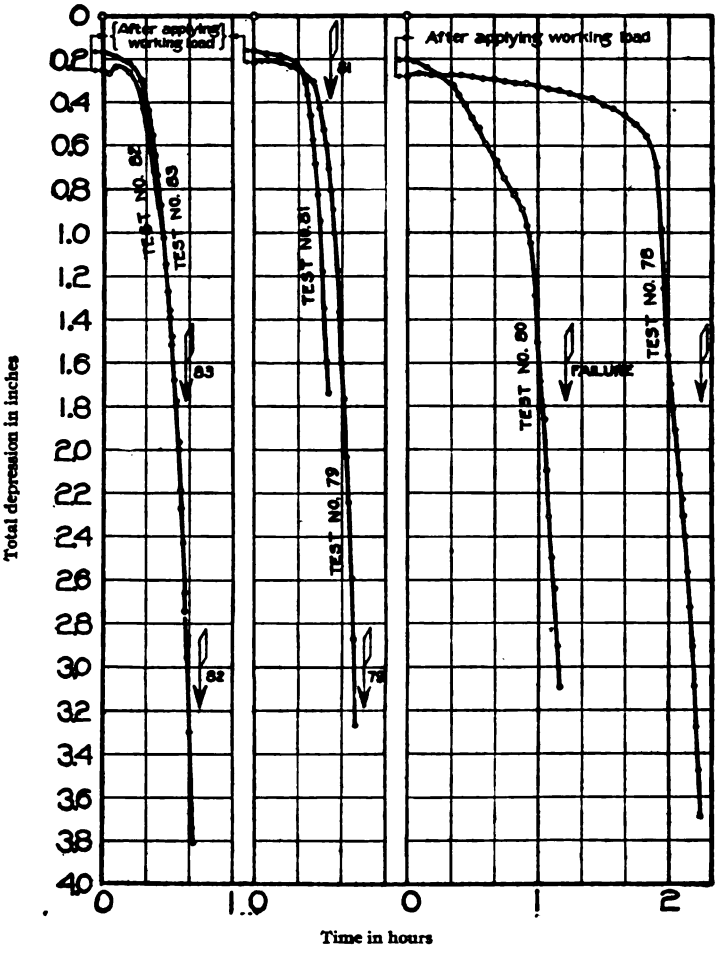

FIG. 47.—*Depression of top of timber columns*

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less than 1 to 15⅜ inches. The smaller deflections obtained in tests where the failure was due to local buckling of individual section members or by direct crushing. The sections that generally failed locally were the plate and channel section and the latticed sections, although the other sections developed this type of failure when exposed to local heating. The large deflections were coincident with more nearly uniform heating and failure with deflection of the column as a whole. The direction of the deflection conformed quite uniformly with the line of least rigidity of the section. The strains at failure caused the thinner plates and section members to buckle between rivet points. The rivets were seldom found to have sheared.

The cast-iron columns usually failed by breaking at one or more points. The curvature of the pieces indicated that deflections of over 9 inches had been sustained before fracture occurred. In a few cases loading was discontinued after the column failed to sustain working load and before fracture occurred, with resultant deflections of 3 to 5 inches.

The failure of the reinforced-concrete columns was due to local crushing and shearing and developed little deflection of the column as a whole.

The failure of the timber columns with the steel plate cap and timber strut bearing was accompanied by lateral slipping of the cap on its bearings due to the softening of the wood immediately adjacent to them. In the case of the timber columns with cast-iron cap and pintle bearing, the immediate cause of failure was fracture of the cap. In both cases the deformation in the wood at the cap, before slipping or fracture of the latter, was so large as to constitute a near equivalent of the ensuing failure.

8. LOG OF FIRE TESTS

The log of the tests includes visual observations during the test, the time of maximum expansion, the time of failure, and effects at failure. The latter was usually accompanied by general or local buckling, the point of maximum lateral deflection being in or near the most highly heated region of the column.

The references after each test give in order the figure number of the corresponding column views, time-temperature curves, the deformation and average temperature curves.

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(a) UNPROTECTED COLUMNS

TEST NO. 1. ROLLED H.—7 mins.—trace of color on flange edges. 8 mins.—maximum expansion. Lateral deflection at middle of column, ⅛ in. to west, increasing to ¾ in. at 10½ mins. 11¼ mins.—failure with buckling to west, maximum at 4½ ft. above base. (Figs. 58, 90, and 146.)

TEST NO. 2. PLATE AND ANGLE.—6 mins.—trace of color on flange edges increasing to cherry red at 15 mins. 17 mins.—maximum expansion. 16 mins.—deflection ⅛ in. to east, increasing to ¼ in. at 18½ mins. 19¼ mins.—failure with buckling to east, maximum at 5½ ft. above base. (Figs. 58, 90, and 146.)

TEST NO. 3. PLATE AND CHANNEL.—10 mins.—trace of color on flanges. 11 mins.—maximum expansion. 14 mins.—failure with buckling to west, maximum at 7½ ft. above base. Column low cherry red in color. (Figs. 58, 90, and 146.)

TEST NO. 4. LATTICED CHANNEL.—9 mins.—maximum expansion. 10 mins.—trace of color on flange edges and web. 11 mins.—failure with buckling to east, maximum at 3½ ft. above base. (Figs. 58, 90, and 146.)

TEST NO. 5. Z BAR AND PLATE.—9 mins.—deflection of ⅛ in. north increasing to ¼ in. north at 11½ mins. 10 mins.—dull red color noted. 11 mins.—maximum expansion. 14¼ mins.—failure with buckling to north, maximum at 6 ft. above base. (Figs. 58, 91, and 146.)

TEST NO. 6. I BEAM AND CHANNEL.—8 mins.—no deflection. 10 mins.—trace of color on flanges. 13 mins.—maximum expansion. 16 mins.—deflection of ¾ in. north. 17 mins.—failure with buckling to north, maximum at 5½ ft. above base. (Figs. 59 and 91.)

TEST NO. 7. LATTICED ANGLE.—10 mins.—trace of color noted in lattice. 11 mins,—maximum expansion. 14 mins.—failure with local buckling in each angle from 8 to 10 ft. above base. (Figs. 59 and 91.)

TEST NO. 8. STARRED ANGLE.—13½ mins.—deflection of ⅛ in. northeast, increasing to ⅝ in. north at 18½ mins. 15 mins.—traces of color increasing to red at 18 mins. 19 mins.—maximum expansion. 21½ mins.—failure with buckling to north, maximum at 5 ft. above base. (Figs. 59 and 91.)

TEST NO. 9. ROUND CAST IRON, ENDS RESTRAINED.—18 mins.—traces of color increasing to bright red at 32 min. 24 mins.—maximum expansion. 26 mins.—deflection of ⅜ in. west, increasing to 1 in. west at 31 mins. 34¼ mins.—failure, buckling to west and breaking at 1½, 5, and 11 ft. above base. Thinnest metal on southeast side. (Figs. 60, 92, and 147.)

TEST NO. 10. ROUND CAST IRON.—20 mins.—dull red color increasing to cherry red at 28 mins. 23 mins.—maximum expansion. 26 mins.—deflection of ¼ in. northwest, increasing to ½ in. west at 31 mins. 34½ mins.—failure, buckling to southwest and breaking about 5 ft. above base. Thinnest metal on northeast side. (Figs. 60, 92, and 147.)

TEST NO. 10A. ROUND CAST IRON.—26 mins.—no color noted. Deflections not measured. 22 mins.—maximum expansion. 34¼ mins.—failure, buckling to north and breaking in two about 5 ft. above base. Column glowing red. (Figs. 60, 92, and 147.)

TEST NO. 11. ROUND CAST IRON, CONCRETE FILLED.—33 mins.—traces of color increasing to dull red at 39 mins. 27 mins.—maximum expansion. 31 mins.—deflection of ⅜ in. west, increasing to 1¼ in. west at 41 mins. 45¼ mins.—failure, buckling to west and breaking in two about 5 ft. above base. Thinnest metal on north side of section. (Figs. 60, 92, and 147.)

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TEST NO. 12. STEEL PIPE, CONCRETE FILLED.—14 mins.—maximum expansion. 21 mins.—traces of color increasing to cherry red at 32 mins. 12½ mins.—deflection of ¼ in. northeast, increasing to ¾ in. northeast at 31 mins. 36 mins.—failure, buckling to northeast, maximum at 6½ ft. above base. (Figs. 60, 93, and 147.)

TEST NO. 13. REINFORCED STEEL PIPE, CONCRETE FILLED.—15 mins.—traces of color increasing to bright red at 38 mins. 52 mins.—maximum expansion. 4 mins.—slight deflection to northwest noted, increasing to ¾ in. at 29 mins., 1⅛ in., at 1 hour, and 2⅛ in. northwest at 1 hr., 8 mins. 1 hr. 11¾ mins.—failure, buckling to northwest, maximum at 6 ft. above base. (Figs. 60 and 93.)

(*b*) PARTLY PROTECTED COLUMNS

TEST NO. 14. ROLLED H, 1:2:4 JOLIET GRAVEL CONCRETE.—No cracks or spalling noted in concrete before failure. 45 mins.—maximum expansion. 25 mins.—slight deflection to west noted, increasing to ¼ in. at 46 mins. 1 hr. 4¼ mins.—failure, buckling to west, maximum at 7 ft. above base. Loud report at failure probably due to failure of concrete in compression.

After failure.—Concrete generally loose, due to large deflection at failure; no marked heat or dehydration effects. Two pieces of concrete filling 3¾ by 7$\frac{1}{16}$ by 10 in. tested on end three weeks after the fire test gave ultimate compressive strengths of 1440 and 1800 lb. per sq. in., respectively. (Figs. 61 and 94.)

TEST NO. 15. ROLLED H, 1:2:4 GRANITE CONCRETE.—25 min.—fine horizontal cracks appeared in concrete on west at center. 30 mins.—cracks on west opening up and extending, column otherwise unaffected; steel dull red. 40 mins.—maximum expansion. No spalling before failure. 24 mins.—slight deflection to west noted, increasing to ¼ in. at 36 mins. 48¼ mins.—failure, buckling to west, maximum at 7½ ft. above base.

After failure.—Concrete generally loosened, this probably occurring at failure. Piece of filling tested 9 days after the fire test had ultimate compressive strength of 960 lb. per sq. in. (Figs. 61 and 94.)

TEST NO. 16. PLATE AND ANGLE, 1:2:4 TRAP CONCRETE.—40 mins.—steel flanges dull red. Maximum expansion. No cracks or spalling noted in concrete before failure. 36 mins.—deflection of ⅛ in. to east. 44½ mins.—failure, buckling to the east, maximum at 5½ ft. above base.

After failure.—Concrete cracked and loose in several places. (Figs. 61 and 94.)

TEST NO. 17. PLATE AND ANGLE, 1:1½:4½ CINDER CONCRETE.—38 mins.—flanges of angles dull red. 40 mins.—maximum expansion. 41 mins.—no cracks or spalling of concrete. 41¾ mins.—failure, buckling to east, maximum at 5 ft. above base.

After failure.—Concrete cracked at points of maximum flexure and had fallen out to a depth of 1 in. in places. (Figs. 61 and 94.)

TEST NO. 18. LATTICED CHANNEL, 1:2:4 TRAP CONCRETE.—15 mins.—traces of color showing in concrete and all surfaces glowing at 30 min. 2 hrs.—maximum expansion. 2 hrs. 45 mins.—several vertical cracks ⅛ in. wide and about 18 in. long on east and west sides, mostly in lower half; concrete bulged out ½ to ¾ in. at cracks. 2 hrs. 53 mins.—failure, with local buckling of channels about 3½ ft. above base.

After failure.—Concrete in channels cracked along flanges in lower half and quite crumbly; concrete in upper half quite firm. A piece of concrete, filling the space between the channels near the middle of the column, 5½ by 9½ by 14 in. was tested on end 34 days after the fire test and developed an ultimate compressive strength of 577 lb. per sq. in. (Figs. 61 and 95.)

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TEST NO. 19. Z BAR AND PLATE, 1:3:5 LIMESTONE CONCRETE.—20 mins.—traces of color in steel increasing to cherry red at 40 min. 50 mins.—concrete flowing. Very little cracking of concrete before failure. 1 hr.—maximum expansion. 35 mins.—deflection of ¼ in. north, increasing to 1⅛ in. at 1 hr. 4 mins. 1 hr. 7¼ mins.—failure, buckling to north, maximum at 7 ft. above base.

After failure.—Concrete generally loose from steel on north and south sides, standing out to ⅞ in. in places. A few horizontal cracks in the concrete were present on the east and west sides. A piece of filling 11¼ in. long tested 14 days after the fire test developed ultimate compressive strength of 730 lbs. per sq. in. (Figs. 62 and 96.)

TEST NO. 20. I BEAM AND CHANNEL, 1:3:5 TRAP CONCRETE.—34 mins.—trace of color on steel flanges. 36 mins.—color in concrete. 40 mins.—flanges dull red. 60 mins.—concrete and exposed steel bright red. 1 hr. 8 mins.—no cracks or spalling of concrete. 1 hr. 10 mins.—maximum expansion. 1 hr. 24½ mins.—failure, buckling to south, maximum at 6 ft. above base.

After failure.—Concrete remained in place except where crushed at middle, top, and bottom. Rivet heads probably helped to hold it. (Figs. 62 and 96.)

TEST NO. 21. I BEAM AND CHANNEL, 1:3:5 TRAP CONCRETE.—38 mins.—traces of color on steel flanges. 1 hr., 10 mins.—maximum expansion. 1 hr. 20 mins.—no cracks or spalling of concrete. Steel flanges dark red, concrete bright red. 1 hr. 21¾ mins.—failure, buckling to north, maximum at 6 ft. above base.

After failure.—Concrete remained in place except where crushed at failure. (Figs. 62 and 96.)

TEST NO. 22. LATTICED ANGLE, 1:2:4 LIMESTONE CONCRETE.—25 mins.—concrete glowing on corners; a number of fine cracks noted on all faces. 1 hr. 40 mins.—maximum expansion. 3 hrs.—surface of column glowing at white heat. 4 hrs. 50 mins.—large vertical cracks appeared under brackets near top. Very little cracking or spalling of concrete before failure. 5 hrs. 14 mins.—failure with local buckling of steel and crushing of concrete about 11 ft. above the base. Unprotected brackets probably caused failure at this point by conducting heat into angles.

After failure.—Except at point of failure concrete did not appear greatly damaged. Limestone calcined to depth of 1 in., and one month after test the whole exterior concrete had become loose due to air slaking of the lime. From temperature of steel at 3 hrs. it is probable that the concrete carried most of the load after this time. (Figs. 62 and 95.)

(c) PLASTER ON METAL-LATH PROTECTIONS

TEST NO. 23. PLATE AND ANGLE, TWO 1-IN. LAYERS OF PORTLAND-CEMENT PLASTER ON EXPANDED METAL LATH.—12½ mins.—distinct thud heard caused by failure of covering due to expansion; plaster cracked and spalled on all faces at top of column near bottom of bracket. 23 mins. to 1 hr. 15 mins.—some six or eight fine vertical and horizontal cracks, 3 to 8 in. long, appeared on east, south, and west sides from 1 to 8 ft. above base. Very little change up to failure, except that all cracks opened up slightly. 2 hrs. 30 mins.—maximum expansion. 2 hrs. 41 mins.—deflection of ⅛ in. northwest, increasing to 1 in. west at 2 hrs. 51 mins. 2 hrs. 52 mins.—failure, buckling to west, maximum at 7½ ft. above base.

After failure.—Plaster of outer layer dehydrated and both outer and inner layers very crumbly. Both layers fairly well keyed to lath. (Figs. 63, 97, and 148.)

TEST NO. 24. PLATE AND CHANNEL, TWO ⅞-IN. LAYERS OF PORTLAND-CEMENT PLASTER ON WOVEN WIRE LATH.—8½ mins.—distinct thud heard; plaster cracked and spalled on all faces at bottom of bracket exposing lath on corners. 14 mins. to 1 hr. 45 mins.—some 15 fine vertical cracks, 3 to 8 in. long, appeared on all sides near corners. 2 hrs.—maximum expansion. 2 hrs. 15 mins.—very little change except some cracks opened up to ⅛ in. 2 hrs. 24 mins.—failure, with local buckling about 6 ft. above base.

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After failure.—Outer layer of plaster dehydrated; inner layer fairly hard except at failure point. Plaster well keyed to lath. (Figs. 63, 97, and 148.)

TEST NO. 25. Z BAR AND PLATE, ONE 1-IN. LAYER OF PORTLAND-CEMENT PLASTER ON EXPANDED METAL LATH.—10 mins.—slight noise heard; plaster cracked and spalled on all sides under bracket. 48 to 54 mins.—fine vertical cracks noted on all faces near corners, opening up slightly toward end of test. 57 mins.—maximum expansion. 41 mins.—slight deflection to north noted, increasing to ¼ in. at 1 hr. 1 min. 1 hr. 7¾ mins.—failure, buckling to north, maximum at 5½ ft. above base.

After failure.—Plaster appeared to be in fairly good condition and was quite hard, except where crushed. Plaster well keyed, covering inner face of lath. (Figs. 63, 98, and 149.)

TEST NO. 26. LATTICED ANGLE, ONE 1⅛-IN. LAYER OF PORTLAND-CEMENT PLASTER ON EXPANDED METAL LATH.—10 to 17 mins.—cracking and some spalling of plaster on bracket. 20 mins. to 1 hr.—some 15 fine vertical cracks, 6 to 12 in. long, appeared near corners on all sides; also horizontal cracks near bottom. Before failure bracket cracks had opened up to ½ in. and others nearly ⅛ in. 1 hr. 10 mins.—maximum expansion. 1 hr. 23½ mins.—failure, with local buckling about 6 ft. above base.

After failure.—Very little strength in plaster, very crumbly. Keys covered lath on inside. ¾ in. air space between plaster and angles. (Figs. 63, 98, and 149.)

TEST NO. 27. ROUND CAST IRON, ONE 1½-IN. LAYER OF PORTLAND-CEMENT PLASTER ON HIGH-RIBBED METAL LATH.—18 mins.—vertical and horizontal cracks under bracket. 1 hr. 40 mins. to 2 hrs. 5 mins.—several fine vertical and horizontal cracks, 3 to 12 in. long, mostly near middle. 2 hrs. 11 mins.—maximum expansion. 2 hrs. 50 mins.—cracks opening up to ⅛ in. 2 hrs. 30 mins.—slight deflection to southwest noted, increasing to ⅛ in. at 2 hrs. 51 mins., and 1¼ in. southwest at 2 hrs. 56 mins. 2 hrs. 58 mins.—failure, buckling to southwest and breaking about 6 ft. above base.

After failure.—Plaster fairly hard where not crushed. Plaster pushed through to iron except at ribs; average thickness of plaster 1½ in. (Figs. 63, 98, and 149.)

(*d*) CONCRETE PROTECTIONS

TEST NO. 28. ROLLED H. 2-IN. 1:2:4 LIMESTONE CONCRETE.—30 to 45 mins.—a few fine vertical cracks at middle and top of east face opening up slightly toward failure. No spalling before failure. 3 hrs. 30 mins.—maximum expansion. 6 hrs. 33¾ mins.—failure, buckling to west, maximum at 6½ ft. above base.

After failure.—Concrete fairly hard although calcined on surface. Flanges exposed at middle and flange edges exposed at several points, this all occurring at failure. Wire tie not broken. (Figs. 64 and 99.)

TEST NO. 28A. ROLLED H. 2-IN. 1:2:4 LIMESTONE CONCRETE. NOT TIED.—2 hrs. 55 mins. to 3 hrs. 23 mins.—several fine vertical cracks 3 to 12 in. long and about 3 in. from corners appeared on east and west faces. No spalling or other effects noted before failure. 3 hrs. 31 mins.—maximum expansion. 5 hrs.—slight deflection to west noted, increasing to ¼ in. at 6 hrs. 2 mins., and 1½ in. west at 7 hrs. 2 mins. 7 hrs. 9¼ mins.—failure, buckling to west, maximum 6¾ ft. above base.

After failure.—Concrete surface fairly firm although discoloration and calcination extend to depth of 1 in. Concrete fell off at failure, exposing about one-half of area of steel flanges. (Figs. 64, 100, and 150.)

TEST NO. 29. ROLLED H. 2-IN. 1:2:4 TRAP CONCRETE.—50 mins. to 3 hrs. 50 mins.—several fine vertical and horizontal cracks appeared on east and west faces, which became larger near failure. No spalling or other effects noted before failure. 3 hrs. 15 mins.—Maximum expansion. 3 hrs. 30 mins.—slight deflection to west noted, increasing to ¼ in. at 4 hrs. 2 mins., and 1½ in. west at 4 hrs. 32 mins. 4 hrs. 38½ mins.—failure, buckling to west, maximum at 5½ ft. above base.

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After failure.—Surface of concrete reddish in color. No fusion noted. Concrete fell off at failure exposing flange for 1½ ft. on south side and flange edges at other points. Wire tie not broken. (Figs. 64, 101, and 151.)

TEST NO. 30. ROLLED H. 2-IN. 1:2:4 JOLIET GRAVEL CONCRETE.—37 mins.—fine cracks noted in brackets. 3 hrs. 4 mins. to 5 hrs. 5 mins.—a number of fine cracks, 2 to 12 in. long, mostly vertical and near edges, appeared on east and west faces opening up to $\frac{1}{16}$ of an in. near end of test in some cases. No spalling before failure. 3 hrs. 45 mins.—maximum expansion. 6 hrs. 40 mins.—slight deflection to east noted, increasing to ⅜ in. at 7 hrs. 9 mins. 7 hrs. 16 mins.—failure, buckling to east, maximum at 5½ ft. above base.

After failure.—Concrete on surface fairly hard after test, but disintegrated in 30 days due to air slaking. Flanges and flange edges exposed in places due to spalling of concrete at failure. (Figs. 64 and 102.)

TEST NO. 31. ROLLED H. 2-IN. 1:2:4 SANDSTONE CONCRETE.—28 mins. to 1 hr. 5 mins.—vertical cracks appeared on east and west faces about 3 in. from edges, running nearly full length of column, opening up in places to ⅛ in,; also a few fine vertical cracks on north and south faces in lower half. 45 mins.—southeast corner spalled off 3 in. deep from 3 to 6 ft. up, exposing edge of steel. 1 hr. 5 mins. to 3 hrs.—vertical cracks at corners became continuous, opening up to $\frac{1}{16}$ in.; concrete on southeast and southwest corners generally spalled off or loose in lower half, exposing edges of flanges in places. Very little change to failure except for minor cracking and spalling. 2 hrs. 46 mins.—maximum expansion. 2 hrs.—slight center deflection to northwest noted, increasing to ⅝ in. at 3 hrs. 1 min., and 2¼ in. northwest at 4 hrs. 11 mins. 4 hrs. 11½ mins.—failure, buckling to west, maximum at 5½ ft. above base.

After failure.—Concrete on flanges disintegrated and crumbly. Nearer the web the concrete was harder but had apparently lost much strength. (Figs. 64, 103, and 152.)

TEST NO. 32. ROLLED H. 2-IN. 1:2:5 CINDER CONCRETE.—3 hrs.—no cracking or spalling noted. 3 hrs. 20 mins. to 3 hrs. 38 mins.—a few small vertical cracks 4 to 6 in. long appeared near corners on east and west faces in lower half extending in length and opening up toward end of test; no spalling before failure. 2 hrs. 32 mins.—maximum expansion. 3 hrs.—slight deflection to east noted, increasing to ½ in. at 3 hrs. 31 mins., and 1⅝ in. east at 3 hrs. 43 mins. 3 hrs. 44 mins.—failure, buckling to east about 6 ft. above base.

After failure.—Concrete very crumbly to a depth of about 1 in. Quite hard and apparently little affected at column web 5 in. from surface. (Figs. 65, 104, and 153.)

TEST NO. 32A. Z BAR AND PLATE. 2-IN. 1:2:5 CINDER CONCRETE.—2 hrs. 50 mins.—maximum expansion. 3 hrs.—no cracking or spalling noted. 3 hrs. 50 mins.—fine vertical cracks developing on all faces mostly near northeast and southwest corners. 3 hrs. 30 mins.—slight deflection to north noted, increasing to ⅜ in. at 3 hrs. 51 mins., and 1⅝ in. north at 4 hrs. 1 min. 4 hrs. 2 mins.—failure, buckling to north, maximum at 5½ ft. above base.

After failure.—Concrete crumbly and dehydrated to a depth of about 1½ in. Beyond this it was harder and apparently little affected. (Figs. 65, 104, and 153.)

TEST NO. 33. ROLLED H. 4-IN. 1:2:4 LIMESTONE CONCRETE.—2 hrs. to 4 hrs.—fine vertical cracks 6 to 12 in. long on east and west faces, 4 in. from corners, in lower half; also small cracks in bracket; cracks opening slightly toward end of test; no spalling. 7 hrs. 50 mins.—maximum expansion. 8 hrs.—column still supporting working load, with no apparent change; less than ⅛ in. deflection. 8 hrs. 2 mins.—load increased, with fire going until failure occurred under load of 431 000 lb., with local buckling about 10 ft. above base, at 8 hrs. 8 mins.

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After failure.—Outer ¼ in. of concrete soft and could be easily knocked off 8 days after test, and was dehydrated for another 1½ in., inside of which it was hard and apparently little affected by the heat. (Figs. 65, 105, and 154.)

TEST NO. 33A. ROLLED H. 4-IN. 1:2:4 LIMESTONE CONCRETE. NOT TIED.—30 mins.—slight surface flaking at several points. 34 to 40 mins.—fine cracks noted on east and west sides of bracket. 2 hrs. 10 mins. to 5 hrs. 20 mins.—three or four fine vertical cracks 3 to 24 in. long on east and west faces, 3 in. from corners, in lower half; no spalling or further cracking before failure. 7 hrs. 50 mins.—maximum expansion. 8 hrs.—column still supporting load with no apparent change; less than ⅛ in. deflection. 8 hrs. 5 mins.—load increased with fire going until failure occurred under load of 405 000 lb. with buckling to the east, maximum 7 ft. above base, at 8 hrs. 7¼ min.

After failure.—In lower 8 ft. concrete checked and pitted with small holes of $\frac{1}{16}$ to $\frac{1}{8}$ in. diameter, and hard to depth of over $\frac{1}{8}$ in. Concrete calcined to depth of 1½ in. from surface. Concrete on flanges broke loose at failure due, in part, to absence of wire tie. (Figs. 65 and 106.)

TEST NO. 34. ROLLED H. 4-IN. 1:2:4 GRANITE CONCRETE.—32 mins. to 1 hr. 5 mins.—fine vertical cracks about 3 in. from corners mostly on east and south sides, 5 to 9 ft. up; also cracks on all sides in bracket. 1 hr. 28 mins.—cracks opened generally $\frac{1}{16}$ to $\frac{1}{8}$ in. width. 2 hrs. 13 mins.—crack through concrete at southeast corner 6 ft. up. 4 hrs. 7 mins.—¼-in. crack through northeast corner 6 ft. up. 6 hrs. 34 mins.—maximum expansion. 6 hrs. 40 mins.—very little change except for slight spalling on northwest corner 8 ft. up. 7 hrs. 33 mins.—spalling, northwest corner, 6 by 3 by 3 in. at 8 ft. up; spalling, southeast corner, 6 by 6 by 20 in. 6½ ft. up, exposing steel. 3 hrs.—slight deflection to east noted, increasing to ⅜ in. at 7 hrs. 31 mins. 7 hrs. 58 mins.—failure, buckling to east, maximum at 7 ft. above base.

After failure.—Incipient fusion of concrete 1½ to 2 in. in depth throughout. Concrete underneath very soft and crumbly. (Figs. 65, 107, and 155.)

TEST NO. 34A. ROLLED H. 4-IN. 1:2:4 GRANITE CONCRETE.—55 mins. to 1 hr. 45 mins.—small vertical cracks appeared on all faces 3 to 5 in. from corners, mostly from 5 to 7 ft. up, opening up to ⅛ in. in some cases; also several cracks in bracket. 3 hrs. 16 mins.—cracks opening up to ¼ in., no spalling except small piece at bracket. 4 hrs. 44 mins.—gas shut off for 1½ mins. to cool furnace. 5 hrs. 50 mins.—maximum expansion. 6 hrs. 30 mins.—gas shut off for 2 mins. to clear furnace; small spall noted on southwest corner 7½ ft. up. 7 hrs.—gas shut off for 2 mins.; incipient fusion apparently present. Minor spalling of corners noted. 6 hrs. 30 mins.—slight deflection to west noted, increasing to ½ in. at 7 hrs. 21 mins. 7 hrs. 23 mins.—failure, buckling to west, maximum at 5 ft. above base.

After failure.—Incipient fusion of concrete 1 to 1½ in. in depth over whole surface. (Figs. 65 and 108.)

TEST NO. 35. ROLLED H. 4-IN. 1:3:5 LIMESTONE CONCRETE.—56 mins.—fine vertical crack, 12 ins. long, on both faces at southeast corner at bottom, 4 in. from edge. 3 hrs.—no spalling or further cracking noted. 4 to 7 hrs.—furnace filled with heavy flame; impossible to see column. 7 hrs. 40 mins.—maximum expansion. 8 hrs.—column still supporting working load with no apparent change; deflection less than ⅛ in.; load increased with fire going until failure occurred under load of 348 000 lbs. with local buckling 7½ ft. above base, at 8 hrs. 7 mins.

After failure.—Surface concrete hard and sand fused throughout to depth of ⅛ in. to ¼ in.; limestone completely calcined up to 1 in. from surface, and soft and dehydrated up to 2½ ins. from surface. Concrete was harder farther in, although less so than in 33 and 33A. (Figs. 66, 109, and 156.)

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TEST NO. 36. PLATE AND ANGLE. 2-IN. 1:2:4 TRAP CONCRETE.—37 mins.—small crack on west face, 5 in. long, running up diagonally from southwest corner, 2½ ft. up. 1 hr. 13 mins.—small spall, 3 by ¼ in. on southeast corner, 6½ ft. up. 1 hr. 20 mins. to 3 hrs. 40 mins.—several fine cracks at bracket. 3 hrs. 10 mins.—maximum expansion. 3 hrs. 46 mins.—several vertical cracks 2 in. from corners on east and west faces, 2 ft. to 4 ft. up; no spalling before failure. 3 hrs.—slight deflection to northwest noted, increasing to ⅞ in. at 3 hrs. 44 mins. 3 hrs. 53¼ mins.—failure, buckling to west, maximum at 6 ft. above base.

After failure.—Concrete soft and crumbly to a depth of 1½ to 2 in. (Figs. 66, 103, and 151.)

TEST NO. 37. PLATE AND ANGLE. 4-IN. 1:2:4 TRAP CONCRETE, ROUND SECTION.—35 mins. to 1 hr.—small cracks noted on all sides at bracket. 1 hr. 25 mins. to 1 hr. 40 mins.—spalling of southwest and southeast corners at bracket. 4 hrs. 20 mins.—no further cracking or spalling. 6 hrs. 10 mins.—maximum expansion. 6 hrs. 24 mins.—spalling across west face at bracket about 1 in. deep. 7 hrs. 10 mins.—spalling south face at bracket; impossible to observe lower part of column on account of furnace gases. 6 hrs. 30 mins.—slight deflection to west noted, increasing to ½ in. at 7 hrs. and 1¾ in. west at 7 hrs. 30 mins. 7 hrs. 34½ mins.—failure, buckling to west, maximum at 5½ ft. above base.

After failure.—Considerable fusion and flowing of concrete to depth of 2 in. and incipient fusion for 1 to 1½ in. farther in, in lower 8 ft. Partial fusion to ½ in. depth above this point. Concrete crumbly for 1 in. inside fused portion; fairly hard farther in. Trap rock extends to surface quite generally in upper portion which was not fused. (Figs. 66, 110, and 157.)

TEST NO. 38. PLATE AND CHANNEL. 2-IN. 1:2:4 JOLIET GRAVEL CONCRETE.—45 mins. to 2 hrs. 55 mins.—several fine vertical cracks appeared on east and west faces about 2½ in. from corners, 5 to 8 ft. above base, some opening up to 1/16 in., also several cracks in bracket. 3 hrs. 10 mins.—maximum expansion. 3 hrs. 26 mins.—horizontal cracks on north face 4 ft. up, ¼ in. wide. 4 hrs. 6 mins.—vertical cracks on east and west faces opening up to ⅛ in.; very few cracks on north and south faces. 5 hrs. 4 mins.—concrete loose on corners 3 ft. up. 5 hrs. 28 mins.—no spalling. 4 hrs.—slight deflection to east noted, increasing to ½ in. at 5 hrs. 21 mins. 5 hrs. 28¾ mins.—failure, buckling to east, maximum at 2½ ft. above base.

After failure.—Concrete calcined to depth of ¾ in. and very crumbly to depth of 2 in. Inside of this point on web side concrete was quite hard and apparently little affected by the heat. (Figs. 66, 111, and 158.)

TEST NO. 39. PLATE AND CHANNEL. 4-IN. 1:2:4 MERAMEC RIVER GRAVEL CONCRETE.—19 to 26 mins.—vertical cracks developed on east and west faces near corners in lower half and at bracket. 26 mins.—2-ft. spall on northwest corner near middle. 28 mins. to 40 mins.—vertical cracks extending upward and opening up. 40 mins.—horizontal crack, 5 ft. up, across north face. 45 mins.—large spall on southwest corner, 7 ft. up. 1 hr. 20 mins.—cracks opening up to ½ in. in some cases. 1 hr. 26 mins.—concrete cracked through northeast corner, this corner spalling off from 4 to 8 ft. up at 2 hrs. 6 mins., exposing edge of steel for 4 ft. 2 hrs. 13 mins.—southwest corner spalled off for 3 ft. near middle exposing steel. 3 hrs.—northwest corner and west side spalled off above middle of column exposing steel on corner for 2 ft. 3 hrs. 20 mins.—maximum expansion. 3 hrs. 41¼ mins.—failure with local buckling 6½ to 8 ft. above base.

After failure.—Concrete inside of spalled or cracked portions was quite hard and split with fracture of aggregate. Aggregate also fractured by cracks produced in the fire test. (Figs. 66 and 112.)

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TEST NO. 40. LATTICED CHANNEL. 2-IN. 1:2:4 TRAP CONCRETE, ROUND SECTION.—5 hrs. 1 min.—maximum expansion. 6 hrs. 56 mins.—no cracking, spalling, or fusion of concrete. 7 hrs. 25 mins.—a number of vertical cracks on east and west sides in lower half, varying from very fine to one ⅛ by 16 in. at southeast, 3 ft. up. No spalling or fusion. 7 hrs. 55 mins.—cracks opening up; the crack noted above open ¼ in.; no spalling, possibly some fusion. 7 hrs. 57 mins.—failure with local buckling about 5 ft. above base.

After failure.—Fusion of concrete from depth of ½ to 1½ in. near failure point. Very little concrete had run. Concrete outside of steel generally disintegrated; between channels it is harder. (Figs. 67, 113, and 159.)

TEST NO. 41. Z BAR AND PLATE. 4-IN. 1:3:5 LIMESTONE CONCRETE.—10 mins.—fine vertical crack, 2 in. long, on west face near north corner 7 ft. up. 1 hr. to 1 hr. 50 mins.—several vertical cracks, 3 to 12 in. long, appeared on west face near corners in middle section of column; some opening up to ⅛ in.; no spalling. 2 hrs. 50 mins.—concrete cracked through for 2-in. length on northwest corner, 5 ft. up. 7 hrs. 30 mins.—maximum expansion; lateral deflection at middle of column less than ⅛ in. 8 hrs.—column still supporting working load with little apparent change; no spalling; load increased with fire going until failure occurred under load of 332 000 lbs., buckling to south, maximum 6 ft. above base, at 8 hrs. 24¼ mins.

After failure.—Concrete reddish in color and dehydrated to depth of 1 in. from surface. Concrete outside of flanges dehydrated and crumbly. (Figs. 67, 114, and 160.)

TEST NO. 42. Z BAR AND PLATE. 4-IN. 1:3:5 LIMESTONE CONCRETE.—1 hr. 15 mins.—small crack on southwest corner at bracket. 2 hrs. 20 mins.—fine vertical crack on west face 4 in. from northwest corner, 4 ft. up. 8 hrs.—maximum expansion; column still supporting working load with little apparent change; no spalling; deflection less than ⅛ in.; load increased with fire going until failure occurred under load of 333 000 lb. with buckling to north, maximum 6 ft. above base, at 8 hrs. 11½ mins.

After failure.—Concrete affected same as in test No. 41. (Figs. 67, 115, and 161.)

TEST NO. 43. I BEAM AND CHANNEL. 2-IN. 1:2:4 SANDSTONE CONCRETE.—13 mins.—vertical crack, 4 in. long, on southeast corner, 5½ ft. up, causing slight spalling at 20 mins. 21 to 30 mins.—vertical cracks appeared on east and west faces near corners, mostly in middle part of column. 30 mins.—southeast corner spalled off to a height of 5½ ft., exposing edge of steel and ties for 4 ft. 33 to 41 mins.—vertical cracks on east and west faces extended nearly full length of column. 45 mins.—southwest corner spalled off 4 to 10 ft. up. 51 mins.—southeast corner spalled off 6 to 9 ft. up. 55 mins.—cracks opening up to ¼ in. in places. 1 hr. 15 mins.—southwest corner spalled off, lower 4 ft. 2 hrs. 50 mins.—maximum expansion. 3 hrs. 5 mins.—concrete checking all over surface. 4 hrs. 11 mins.—failure, buckling to north, maximum 6 ft. above base.

After failure.—Channel flanges and ties on south oxidized. Concrete discolored on surface and very crumbly for depth of 1 in., the rest dehydrated and possessed little strength. Fractures split aggregate, which also seems to have lost much of its strength, crumbling under light hammer blows. (Figs. 67, 116, and 152.)

TEST NO. 44. I BEAM AND CHANNEL. 2-IN. 1:3:5 SANDSTONE CONCRETE.—17 mins.—vertical crack on northeast corner, 2 to 5 ft. up, corner spalling off at 18 mins., also vertical crack on northeast corner, 3 to 7 ft. up, corner spalling off at 25 mins. 22 mins.—vertical cracks on both sides of northwest and southwest corners. 32 mins.—southeast corner spalled off in lower 3 ft. 35 mins.—vertical cracks on all faces about 2 in. from corners running full length of column. Edges of steel exposed for lower 7 ft. on northeast and southeast corners; concrete loose on northwest and southwest corners. 36½ mins.—southwest corner spalled from 6 to 7 ft. up. 1 hr. 2 mins.—spall east side near middle for length of 3 ft., exposing flange edges. 1 hr. 23 mins.—

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all corners at bracket spalled or cracked. 1 hr. 24 mins.—northwest corner spalled from 6 to 9 ft. up. 1 hr. 35 mins.—southeast flange of channel dull red at middle of column; no other change. 2 hrs. 23 mins.—maximum expansion. 3 hrs. 4¼ mins.—failure, buckling to north, maximum 6 ft. above base.

After failure.—Concrete generally very soft and crumbly. Falls off column readily in large pieces. (Figs. 67 and 116.)

TEST NO. 45. STARRED ANGLE. 2-in. 1:2:4 MERAMEC RIVER GRAVEL CONCRETE, ROUND SECTION.—20 to 30 mins.—a number of deep cracks, mostly vertical, in lower half of column; concrete becoming loose in places. 35 mins.—some spalling near middle of column. 36 mins.—cracks in upper half of column; practically all sides have large cracks. 38 mins.—crack on west, 3 ft. up, open 3 in. exposing steel. 44 mins.—spalling to steel on west. 47 mins.—spalling to steel on north and west near bottom. 55 mins.—wire tie appeared to be holding concrete between angles in place. 56 mins.—edge of steel exposed on west from 1 ft. to 6 ft. up, on north from 1 to 2 ft. up. 1 hr. 7 mins.—steel exposed on south, 1 to 2 ft. up. 1 hr. 10 mins.—large pieces spalled on north side near middle of column. 1 hr. 34 mins.—concrete on lower part of bracket spalled off. 1 hr. 41 mins.—maximum expansion. 1 hr. 47 mins.—failure by local buckling about 3 ft. above base.

After failure.—Coarse aggregate was quite generally broken on fracture planes. Concrete inside of fractures appeared to be in good condition. (Figs. 66 and 112.)

TEST NO. 46. LATTICED ANGLE. 2-in. 1:2:4 TRAP CONCRETE.—1 hr.—diagonal crack, southeast corner at bracket. 2 hrs. 35 mins.—vertical crack, 3 in. long, on east face, 4 in. from southeast corner, 8 ft. up. No spalling before failure. 4 hrs. 55 mins.—maximum expansion. 6 hrs. 43½ mins.—failure with local buckling about 6 ft. above base, each angle deflecting outward about 2½ in.

After failure.—Incipient fusion on surface of concrete with small cracks. Concrete crumbly from dehydration to 1½ in. from surface. It was otherwise apparently in fair condition, except where crushed at point of failure. (Figs. 67 and 117.)

TEST NO. 47. ROUND CAST IRON. 2-in. 1:2:5 CINDER CONCRETE. NOT TIED.—30 mins.—piece of concrete 2 in. diameter, spalled off under bracket on south, exposing bracket; also crack 6 in. long extended down from spall. 2 hrs. 3 mins.—maximum expansion. 2 hrs. 42 mins.—vertical cracks noted on all sides at middle of column. 2 hrs. 47 mins.—concrete fell off from 2 to 10 ft. up, exposing metal. 2 hrs.—slight deflection to south noted, increasing to ¼ in. at 2 hrs. 20 mins. 2 hrs. 48¾ mins.—failure with buckling to south, maximum 6 ft. above base.

After failure.—Column broken up and thickness found to vary from ½ to 1$\frac{1}{16}$ in., with thinnest metal on north. A short horizontal crack had formed on north side 3 ft. above base. Concrete crumbly for ⅜ in. from surface. Inside of this it appears almost unaffected by the fire test. (Figs. 67, 101, and 158.)

(e) HOLLOW CLAY-TILE PROTECTIONS

TEST NO. 48. ROLLED H. 2-IN. SEMIFIRE-CLAY TILE, NEW JERSEY DISTRICT. NO FILLING.—7 mins.—a number of vertical joints open about $\frac{1}{16}$ in. 12 mins.—vertical cracks in center of north, east, and south sides, 5 to 9 ft. up; cracks in about ⅓ of all vertical joints. 20 mins.—vertical cracks extending and in some cases opening to ¼ in.; cracks on all sides at bracket, and lower edge of tile spalled off on southwest corner at bracket. 30 mins.—north face cracked, 2 to 10 ft. up; vertical cracks on east from 3 to 10½ ft., on south from 5 to 9 ft. up; cracks in some cases open ⅜ in. 1 hr. 15 mins.—outer shell of tile spalled off along lower west edge at bracket; little change in tile below; no spalling below bracket before failure. 1 hr. 32 mins.—maximum expansion. 1 hr. 40 mins.—slight deflection to northwest noted, increasing to ⅝ in. at 1 hr. 49 mins. 1 hr. 50 mins.—failure, buckling to west, maximum 8 ft. above base.

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After failing.—At failure tile fell off from 5 ft. up to top course of bracket. Lower two or three courses almost intact. (Figs. 68, 118, and 162.)

TEST NO. 49. ROLLED H. 4-IN. SEMIFIRE-CLAY TILE, NEW JERSEY DISTRICT. NO FILLING.—3½ to 8½ mins.—a number of vertical cracks in tile and joints in lower 8 ft. opened $\frac{1}{16}$ to $\frac{1}{8}$ in. 12 to 24 mins.—cracks opening ¼ to ⅜ in. maximum; west face bulged out ¾ in. from 4 to 8 ft. up. 22 mins.—ties down at 9th and 12th courses. 25 mins.—tile in middle section of column on south and west sides cracked and crushed. 1 hr. 27 mins.—cracks opening up to ⅞ in. in places; no extension of cracks or spalling. 1 hr. 20 mins.—slight deflection to north noted, increasing to ¼ in. at 1 hr. 38 min. 1 hr. 21 mins.—maximum expansion. 1 hr. 40 mins.—failure with compound buckling to northwest 8 ft. above base and to southeast 6 ft. above base.

After failure.—A few tile intact, without cracks; tile generally cracked longitudinally through the shells, not often cracked transversely across webs. This probably accounts for the fact that no spalling of shells occurred until very near failure. (Figs. 68, 118, and 162.)

TEST NO. 50. PLATE AND ANGLE. 2-IN. SURFACE-CLAY TILE, BOSTON DISTRICT. GRANITE CONCRETE FILL. (TILE, 6 IN. WIDE, LAID HORIZONTALLY.)—7 mins.—cracking and spalling on corners, 4 to 6 ft. up. 10 mins.—tile spalled, concrete exposed on south and east for 1 ft. near bottom. 16 mins.—tile bulged out on south face from bottom to 6½ ft. up. 21 mins.—tile fell off on south where bulged, exposing steel. 26 mins.—concrete exposed from 4 to 6½ ft. up on north, east, and west. 32 mins.—cracks opening up at joints in upper half. 55 mins.—maximum expansion. 1 hr. 5¾ mins.—failure, with local buckling about 5½ ft. above base.

After failure.—Tile broke clear from concrete with no splitting of tile at keys. Transverse shearing of long tile at joints of short tile very marked. Tile in all courses fell in whole or part except top bracket course and three lower courses. Concrete in fill where not crushed by compression apparently uninjured. (Figs. 68 and 119.)

TEST NO. 50A. PLATE AND ANGLE. 2-IN. SURFACE-CLAY TILE, BOSTON DISTRICT. GRANITE CONCRETE FILL. (TILE 6 IN. WIDE, LAID HORIZONTALLY.)—5 mins.—several horizontal joints open ⅛ in.; vertical crack 12 in. long through tile on east face, 6¾ ft. up; large number of vertical cracks near corners at bracket. 7 mins.—part of one outer shell on each of south and west, near middle spalled. 13 mins.—vertical cracks ¼ in. wide opposite joints at all corners. 13 to 20 mins.—outer shell spalled off for 2 ft. at middle on east and west; outer shell bulged out ½ in. in several places. 21 mins.—edge of steel exposed for 6 in. on northeast corner, 6 ft. up. 21 to 39 mins.—tile fell off, exposing concrete for 3 ft. at middle on east and west; buckling of tile increasing on flanges to ¾ in. maximum; upper 3 ft. intact, except for ⅛-in. cracks at bracket. 1 hr. 14 mins.—tile fell off, exposing concrete in lower 3 ft. on west face; otherwise little change. 1 hr. 24 mins.—mortar joints evidently not very full on north and south, can see through in places. 1 hr. 42 mins.—tile buckled out 1 in. on north, 5 ft. up. 1 hr. 44 mins.—maximum expansion. 1 hr. 59½ mins.—failure, buckling to west, maximum 5¼ ft. above base.

After failure.—All tile fell off at failure except parts of courses in lower foot and in upper 3 feet. Concrete fully fills reentrant portions except where crushed out at point of failure. (Figs. 69, 119, and 163.)

TEST NO. 51. PLATE AND ANGLE. 4-IN. SURFACE-CLAY TILE, BOSTON DISTRICT. GRANITE CONCRETE FILL.—7 to 17 mins.—considerable cracking and bulging, mostly in middle courses, where some corners were shattered. 14 mins.—vertical cracks in about 50 per cent of tile. 18 to 38 mins.—bulging increasing, vertical cracks opening to ¾ in. maximum; minor spalling on southeast and southwest corners in middle courses. 20 mins.—ties on 2d and 8th courses down. 36 mins.—tie on 6th course down. 40 to 52 mins.—outer shell of tile spalled off from 5 to 7 ft. up on south and

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west. 1 hr. 22 mins.—bulging of tile increased to 1½ in. maximum. 1 hr. 47 mins.—tile fell on north, to 7 ft. up, exposing mortar and concrete. 1 hr. 30 mins.—slight deflection to west noted, increasing to ⅛ in. at 2 hr. 2 hrs. 11 mins.—maximum expansion. 2 hrs. 17¼ mins.—failure, buckling to west, maximum 6 ft. above base.

After failure.—Tile generally shattered. Parts of tile near bottom blackened and partly fused by the heat. Concrete fills web spaces fairly; it is apparently little injured. Mortar joint on flanges not full in places, but voids are partly filled with concrete. (Figs. 69 and 120.)

TEST NO. 51A. PLATE AND ANGLE. 4-IN. SURFACE-CLAY TILE, BOSTON DISTRICT. GRANITE CONCRETE FILL.—3 to 15 mins.—considerable cracking and some bulging at corners. 16 mins.—cracks quite general, open to ¼ in. in lower two-thirds, not over ⅛ in. above. 16 to 44 mins.—cracks increasing and opening up; bulging out of tile at horizontal joints to maximum of 2 in., mostly in middle four courses. 38 mins.—tie broken on 5th course. 1 hr. 6 mins.—cracks and bulges opening up in lower 8 ft.; little change above. 1 hr. 14 mins.—ties dropped on 2d and 3d courses. 1 hr. 20 mins. to 1 hr. 32 mins.—outer shells on east and west sides in middle courses spalling; at end of this period tile had fallen, exposing concrete from 4 to 8 ft. up on east and from 4 to 10 ft. up on west. 1 hr. 44 mins.—all ties down except on three upper courses. 1 hr. 45 mins.—tile buckled away from flanges on north and south 6 ft. up. 2 hrs. 6 mins.—tile spalled on north 6 ft. up, exposing east flange 3 by 4 in. 2 hrs. 23 mins.—tile buckled away from flanges on north and south 8½ ft. up. 2 hrs. 26 mins.—no change in upper courses. 2 hrs. 40 mins.—maximum expansion. 2 hrs. 54 mins.—tile fell on south 6 ft. up, exposing steel. 2 hrs. 41 mins.—lateral deflection at center less than ⅛ in. 2 hrs. 55½ mins.—failure with buckling to west, maximum at 5½ ft. above base.

After failure.—Upper 2½ courses almost intact except for spalling of part of outer shell on west. Tile generally cracked through shells parallel with webs, but some were also cracked through webs parallel with shells. Filling good, no voids. Tile fused near bottom on south and west sides, some pieces had been nearly plastic. Partial fusion extended up to the 10th course. (Figs. 69, 120, and 163.)

TEST NO. 52. PLATE AND CHANNEL. 2-IN. OHIO SHALE TILE. CINDER-CONCRETE FILL.—2 mins.—cracks ⅛ in. wide on east and west. 8 to 11 mins.—cracks increasing and open to ¼ in., some shells buckling, two outer shells spalled, and considerable number of outer shells loose. 16 mins.—cracks open to 1 in. maximum; tie broken on 7th course. 18 mins.—outer shell 7th course on west spalled off. 27 mins.—tie broken on 9th course. 32 mins.—outer shell 9th course on south spalled; a number of loose outer shells held in place by ties. 53 mins.—all tile cracked, but little further spalling. 1 hr. 6 mins.—edge of southeast flange exposed opposite 6th course. 1 hr. 25 mins.—inner shell ready to fall on 7th course, west, edges of flanges probably exposed. 1 hr. 32 mins.—maximum expansion. 1 hr. 40¾ mins.—failure with local buckling about 6 ft. above base.

After failure.—Concrete fill without voids; mortar joints on north and south sides fairly full. Bracket courses almost intact. (Figs. 70, 121, and 163.)

TEST NO. 53. PLATE AND CHANNEL. 4-IN. OHIO SHALE TILE. CINDER-CONCRETE FILL.—2 to 15 mins.—cracking very pronounced on all faces, cracks opening up to 1 in. maximum at end of this period. 7 to 30 mins.—spalling of outer shells beginning at corners and later across faces; outer shells generally loose. 8 to 23 mins.—bulging out of tile at horizontal joints on east and west, 3 to 9 ft. up, increasing to 1½ in. at end of period. 24 mins.—ties off on 3d, 7th, and 9th courses. 30 to 33 mins.—all outer shells spalled off on west, 2 to 6 ft. up on east, 9 to 10 ft. up. 33 to 37 mins.—tile fell on west, 4th and 5th courses, on east 7th, 8th, and 9th courses. 42 mins.—all ties down except on 1st, 8th, 11th, and 12th courses. 45 mins.—tile bulged out on south, exposing steel from 2 to 5 ft. up. 58½ mins.—tile now fallen on west from 3d to 9th

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courses, inclusive. 1 hr. 4 mins.—bracket courses nearly intact. 1 hr. 17 mins.—maximum expansion. 1 hr. 22¼ mins.—failure with local buckling about 7½ ft. above base.

After failure.—Cracks through tile webs parallel with faces predominate, although many units cracked through both faces. Mortar joints generally full. Concrete fill good, no voids. (Figs. 70, 121, and 163.)

TEST NO. 54. LATTICED CHANNEL. 2-IN. OHIO SEMIFIRE-CLAY TILE. TRAP CONCRETE FILL.—2 to 15 mins.—considerable cracking, both in joints and in tile, cracks opening up generally to ⅛ in. at end of period; outer shells spalling near corners and beginning to bulge out at horizontal joints, mostly in middle courses. 24 mins.—bulging out of outer shells increased to maximum of 2 in. 28 mins.—tie broken on 9th course. 48 mins.—nearly all of outer shells spalled off at bracket on south. 50 mins. to 1 hr. 4 mins.—outer shell of tile spalled on south, 4 to 5 ft. up, north half of east face, 3 to 5 ft. up, and on west 7 to 9 ft. up. 1 hr. 22 mins.—tie broken on 4th course. 1 hr. 41½ mins.—inner shells on east fell 4 to 6 ft. up, exposing steel. 1 hr. 43 mins.—inner shells on north fell 4 to 6 ft. up, exposing steel; outer shell spalled on north 3 to 4 ft. up. 1 hr. 52 mins.—tile fell on west 2 to 4 ft. up, exposing flange edges; on north, 3 to 6 ft. up, exposing part of flanges and lattice. 2 hrs. 30 mins.—all tile off on west, 9th and 10th courses. 2 hrs.—slight defection to northwest noted, increasing to ⅛ in. at 2 hrs. 41 mins. and to 1 in. west at 3 hrs. 1 min. 2 hrs. 20 mins.—maximum expansion. 3 hrs. 17¼ mins.—failure, buckling to west, maximum 6 ft. above base.

After failure.—Concrete between flanges of channels quite crumbly; that between channel webs hard and apparently little injured except where crushed. Concrete fairly fills space between steel and tile on both flange and web sides. (Figs. 70, 122, and 164.)

TEST NO. 55. Z BAR AND PLATE. 2-IN. OHIO SEMIFIRE-CLAY TILE. LIMESTONE CONCRETE FILL.—4 mins.—cracking and bulging out of outer shells. 13 to 23 mins.—outer shells generally cracked to ⅜ in. maximum; bulging out at horizontal joints increasing; some spalling at corners. 34 mins.—parts of shell on west spalled at 4 to 5 ft. and 2 to 3 ft. up. 45 mins.—all cracks opening. 1 hr. 4 mins.—wire tie broken on 3d course. 1 hr. 48 mins.—tile fell on west, 7th to 10th courses, inclusive, also part south. 2 hrs.—outer shell spalled on south at bracket course exposing edge of bracket steel. 2 hrs. 8 mins.—outer shell spalled on north, 7th to 9th course. 2 hrs. 37 mins.—tile bulged out 1¼ in. on south 9 ft. up. 2 hrs. 51 mins.—maximum expansion. 3 hrs. 20 mins.—tile fell on west 11th course. 3 hrs. 23 mins. to 3 hrs. 27 mins.—outer shells spalled on north at bracket and from 2 to 6 ft. up; on east, shells now spalled from 2 to 7 ft. up. 3 hrs. 41 mins.—deflection of ⅛ in. south. 3 hrs. 46¾ mins.—failure, buckling to south, maximum at 8 ft. above base.

After failure.—Tile shells remaining on are firmly held. Concrete filled interior fully, including space between Z bar flanges and tile. Concrete hard where tile remained in place; where exposed concrete was calcined to depth of about ½ in. (Figs. 71, 122, and 164.)

TEST NO. 56. Z BAR AND PLATE. 4-IN. OHIO SEMIFIRE-CLAY TILE. LIMESTONE-CONCRETE FILL.—5 to 30 mins.—considerable cracking and some spalling of parts of outer shells; spalling principally in upper half near southwest corner, at middle near northwest corner, and on lower courses at northeast corner. 57 mins.—outer shell bulged slightly at some horizontal joints, most on west at 6 and 7 ft. up; no inner shells spalled. 2 hrs. 30 mins.—maximum expansion. 2 hrs. 55 mins.—outer shell spalled on south at bracket; otherwise little cracking or spalling in last 2½ hrs.; wire mesh in horizontal joints holding tile in place quite effectively. 1 hr. 51 mins.—

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deflection of ⅛ in. south; deflection changed to ⅛ in. northeast at 3 hrs. 1 min., increasing to ⅝ in. at 3 hrs. 21 mins. 3 hrs. 33½ mins.—failure, buckling to north, maximum at 8½ ft. above base.

After failure.—Wire mesh in all horizontal joints lapping at corners. Concrete fill good, no voids; concrete hard, almost intact. (Figs. 71, 123, and 165.)

TEST NO. 57. I BEAM AND CHANNEL. 4-IN. SURFACE-CLAY TILE, CHICAGO DISTRICT. LIMESTONE-CONCRETE FILL.—3 to 13 mins.—general cracking and bulging of a few outer shells, cracks open to ⅛ in. 23 mins.—bulging increasing; general spalling of small pieces from corners. 25 mins.—parts of outer shell spalled on east and west 4 to 5 ft. up. 32 mins.—cracks and bulges generally open ½ in. on all sides from 3 to 6 ft. up. 38 mins.—outer shells loose or fallen on west 5 to 7 ft. up and on south 4 to 6 ft. up. 53 mins.—all outer shells spalled on east and south 3 to 6 ft. up, and on north 5 to 6 ft. up; tile in 5th course on south fell, exposing part of flange. 1 hr. 2 mins.—large amount of tile fell on all sides, exposing most of concrete from 3 to 10 ft. up. 2 hrs. 15 mins.—maximum expansion. 2 hrs. 37 mins.—tile down on all sides from 2 to 12 ft. up, exposing concrete. 2 hrs.—slight deflection to north noted, increasing to ¼ in. at 2 hrs. 50 mins., and to ½ in. at 3 hrs. 21 mins. 3 hrs. 23 mins.—failure, buckling to north, maximum at 8 ft. above base.

After failure.—A number of tile sheared at plaster key, leaving latter in the concrete. Concrete fill fairly full, except at a few points between tile and channel flange. Concrete calcined to depth of ½ in. where exposed; hard where protected. (Figs. 71 and 123).

TEST NO. 58. I BEAM AND CHANNEL. 2 LAYERS OF 2-IN. SURFACE-CLAY TILE, CHICAGO DISTRICT. TILE FILL. WIRE MESH IN HORIZONTAL JOINTS.—3 to 15 mins.—outer shells and tile spalled at corners quite generally. 16 mins.—outer shell of outer tile bulged out 1½ in. at horizontal joint on east 5 ft. up. 26 mins.—part of inner shell of outer layer of tile at southeast corner 10 ft. up fell; outer shells continuing to spall at corners and on parts of some faces. 44 to 48 mins.—outer shells of outer tile spalled on all sides at bracket, also partly on east at 10 ft. up, and all on west 10 to 11 ft. up; outer shell bulged 1 in. at 10½ ft. up on south. 1 hr. 36 mins.—all outer shells now spalled on east from 9 ft. to top. 1 hr. 57 mins.—outer tile down on south, 10th course, and on east, 9th course. 3 hrs.—little change in 1 hr. 3 hrs. 30 mins.—outer shell of outer tile spalled on south, 5th course. 3 hrs. 43 mins.—bracket steel exposed for 6 in. on northeast. 3 hrs. 45 mins.—all tile down in upper two courses on east; on north and south, outer tile down in upper two courses and inner tile bulging out 1 in.; all of bracket exposed on both sides. 3 hrs. 56 mins.—50 to 75 per cent of outer shells of outer tile now down, most of which occurred before 1 hr. 57 mins.; total outer tile and part of inner tile fallen in a few places as noted above; little spalling now taking place. 4 hrs.—maximum expansion. 4 hrs. 25 mins.—no decided change; no fusion noted, although tile at 4 ft. up concave outward as if beginning to fuse. 3 hrs.—slight center deflection to south noted, increasing to ¼ in. at 4 hrs. 31 mins. 4 hrs. 35¼ mins.—failure, with local buckling to south and west about 11 ft. above base.

After failure.—Generally all but outer shell of outer tile in place in lower eight courses. On 9th course outer tile down and almost all tile down in 10th, 11th, and 12th courses. Wire mesh found in all joints except between the 1st and 2d courses, between the 9th and 10th on north and south where the pipes interfered, and between the 11th and 12th where the bracket angles interfered. Some mesh was found between the 10th and 11th courses, although so much tile was down that it was impossible to determine whether all pieces had been placed. Very decided fusion of tile occurred in lower 4 ft. and incipient fusion up to 7 ft. above base, being most pronounced on south side of column. (Figs. 71, 124, and 165.)

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TEST NO. 59. I BEAM AND CHANNEL. 2 LAYERS OF 2-IN. SURFACE-CLAY TILE, CHICAGO DISTRICT. TILE FILL. OUTSIDE WIRE TIES.—3 to 14 mins.—considerable cracking of outer shells on all sides. 14 mins.—outer tile bulged out 1½ in. on west, 7 ft. up. 15 to 23 mins.—spalling of two outer shells at corners. 17 mins.—outer tile bulged out ½ in. on north, 4 ft. up, increasing to 2 in. at 27 mins. 26 mins.—nearly all of outer shells of outer tile spalled on west, 6 to 8 ft. up; tie broken on 8th course. 29 mins.—outer shell spalled on east, 7th and 8th courses. 33 mins.—all outer tile 8th and 9th courses down and part of tile in 7th course. 35 mins.—inner tile bulged out 2½ in. on west, 7 ft. up. 36 mins.—outer tile loose on west in 11th and 12th courses, held by wire. 38 mins.—all outer tile down on south from 3 to 6 ft. up, on north from 3 to 8 ft. up. 42 mins.—inner tile on south bulged out ¾ in., 5 ft. up. 1 hr. 6 mins.—inner tile down on south, 6 to 9 ft. up, exposing both flanges. 1 hr. 16 mins.—all outer tile and parts of inner tile down except for parts of upper and lower courses. 1 hr. 23 mins.—all tile down on west, 6 to 8 ft. up, exposing channel. 1 hr. 25 mins.—maximum expansion. 1 hr.—slight center deflection to north noted, increasing to ⅜ in. at 1 hr. 31 mins. 1 hr. 33¾ mins.—failure with buckling to north, maximum at 7½ ft. above base, and twisting about 22°.

After failure.—All but filling tile down in middle six courses. Filling tile also thrown off near middle of column. All tie wires broken before or at failure. (Figs. 72 and 124.)

TEST NO. 60. LATTICED ANGLE. 2-IN. OHIO SEMIFIRE-CLAY TILE. TRAP-CONCRETE FILL, PLACED BEFORE TILE WAS SET.—3 to 18 mins.—cracking of tile on all sides; verical crack on north near west corner open to ¾ in., 3 to 6 ft. up. 13 to 18 mins.—ties broke on 9th, 6th, 8th, 4th, and 3d courses, in the given order. 18½ mins.—part outer shells spalled on south 3 to 6 ft. up. 26 mins.—tile down on west from 1 to 7 ft. up, exposing concrete. 32 mins.—tile bulged out 2 in. on north 2 to 6 ft. up. 33 mins.—tile down on north from 1 to 6 ft. up. 36 mins.—tile bulged out 1½ in. on east, 2 to 7 ft. up. 38 mins.—a few cracks in tile in upper three courses. 40 mins.—all tile down on east 1 to 10½ ft. up. 48 mins.—tile down on north, 7¼ to 10½ ft. up. 1 hr. 2 mins.—tile down on south, 7th course. 1 hr. 47 mins.—maximum expansion. 1 hr. 58 mins.—little change during last hour. 2 hrs. 7 mins.—all tile down on south up to bracket. 2 hrs. 13 mins.—$\frac{1}{16}$-in. vertical cracks in concrete halfway up increasing to ⅛ in. at 2 hrs. 24 mins. 2 hrs. 21 mins.—center deflection of ⅛ in. northwest, increasing to ¾ in. at 3 hrs. 7 mins. 3 hrs. 9½ mins.—failure with local buckling 4½ to 5 ft. above base.

After failure.—Tile quite generally cracked vertically through both shells, transverse cracks along webs not general. Mortar evidently did not bond tile and concrete. Except at point of failure, cracks in concrete are not wider than $\frac{1}{16}$ in. Concrete very crumbly outside lattice, quite hard inside. (Figs. 72, 125, and 166.)

TEST NO. 61. LATTICED ANGLE. 2-IN. OHIO SEMIFIRE-CLAY TILE. NO FILLING.—3 to 18 mins.—pronounced cracking and bulging especially at middle courses; cracks mostly vertical, opening to 1¼ in. maximum on south, 5 to 7 ft. up; very little spalling. 13 mins.—ties broken on 4th, 6th, and 7th courses. 26 mins.—both outer and inner shells fell on northwest corner 4 ft. up for width of about 4 in. 32 mins.—similar spall on southeast corner 4 ft. up. 35 mins.—considerable bulging of tile on south, 3 to 4 ft. up. 44 mins.—cracks and bulging increased slightly, none now more than ¼ in. 45 mins.—maximum expansion. 47 mins.—center deflection of ¼ in. east. 50¼ mins.—failure with local buckling about 6 ft. above base.

After failure.—Tile cracked transversely in some cases; greater number cracked vertically through both faces without transverse web cracks. (Figs. 73 and 125.)

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TEST NO. 62. ROUND CAST IRON. 2-IN. POROUS SEMIFIRE-CLAY TILE, NEW JERSEY DISTRICT; NO FILLING.—Air pressure tanks shut off, load applied by water pressure only. 6 to 20 mins.—a number of vertical cracks in tile and in mortar joints, opening to maximum of ¼ in. at end of this period. 45 mins.—cracks now in 40 to 50 per cent of tile units, width varying from fine to ⅜ in. in column proper and to ½ in. at bracket. 2 hrs.—all ties in place; cracks have opened up slightly. 2 hrs. 25 mins.—horizontal cracks on west between bracket courses about $\frac{1}{16}$ in. wide. 2 hrs. 42 mins.—similar cracks on southwest, ½ in. wide. 3 hrs.—maximum expansion. 3 hrs. 26 mins.—horizontal bracket cracks open to 1 in. maximum. 4 hrs. 10 mins.—cracks open 1 in. at middle of column; no spalling or bulging of tile. 4 hrs.—slight deflection to southeast noted, increasing to 1⅜ in. at 4 hrs. 10 mins. 4 hrs. 11½ mins.—failure, column unable to support working load. 4 hrs. 13 mins.—load of about 75 000 lbs. held. 4 hrs. 14 mins.—tile fell on east; load of about 25 000 lbs. held. 4 hrs. 14¾ mins.—gas shut off; column buckled to southeast maximum at 8 ft. above base.

After failure.—Column did not crack although metal buckled near surface on compression side at failure point. Cracks in tile are generally vertical, extending through both shells. Tie wires were greatly oxidized and had little strength. (Figs. 73, 126, and 167.)

TEST NO. 63. ROUND CAST IRON. 2-IN. POROUS SEMIFIRE-CLAY TILE, NEW JERSEY DISTRICT; NO FILLING.—6 mins.—vertical crack on east, 4 to 5 ft. up, $\frac{1}{16}$ in. wide. 8 to 11½ mins.—gas shut off accidentally during this period. 13½ to 23½ mins.—fine vertical cracks 2 to 6 ft. long opening up to ¼ in. maximum. 32 mins.—outer shell loose on 3d course, east side. 35 mins.—cracks open to ½ in. 47½ mins.—cracks open to ⅞ in. maximum. 1 hr. 25 mins.—cracks opened but slightly during past 30 mins. 1 hr. 37 mins.—some flaking of outer shells; no spalling. 1 hr. 55 mins.—very little change; tile above 10 ft. up almost intact except for a few cracks ⅛ in. wide. 2 hrs. 20 mins.—maximum expansion. 2 hrs. 23 mins.—about ⅓ of outer shell spalled on 3d course, east side. 2 hrs. 24 mins.—bulging on southeast, 4 to 6 ft. up, with spalling of parts of both shells leaving crack ¾ to 2 in. wide. 2 hrs. 41 mins.—slight center deflection to north, increasing to 1¼ in. at 2 hrs. 55 mins. 2 hrs. 57½ mins.—failure with buckling to north, maximum at 5½ ft. above base.

After failure.—Most cracks in tile were vertical, straight through both shells. Tie wires greatly oxidized, leaving $\frac{1}{16}$ in. effective diameter. At failure point 5½ ft. up, cast iron was mushroomed out about 1 in. for one-half the circumference in a height of 2½ in. Numerous vertical cracks in the iron formed in this region. Tension breaks extend for one-half of the circumference on south, 3½ ft. above the base and 1¾ ft. below the head. The thickness of metal at the center break varied from ⅛ to 1¼ in., the thinnest metal being on the south. (Figs. 73, 126, and 167.)

TEST NO. 76. ROLLED H. 2-IN. HOLLOW CLAY TILE COVERED WITH ¾-IN. LAYER OF GYPSUM PLASTER; LIMESTONE CONCRETE FILL. UPPER 4 COURSES, OHIO SHALE; MIDDLE 4 COURSES, OHIO SEMIFIRE CLAY; LOWER 4 COURSES, SEMIFIRE CLAY, NEW JERSEY DISTRICT.—1 to 3 mins.—outer coat of plaster peeled off and shattered at end of this period; vertical cracks extending through to tile in places. 10½ mins.—inner coat of plaster fell on south, exposing tile 3 to 8 ft. up. 11 to 32 mins.—inner coat of plaster fell, exposing about two-thirds of tile at 21 mins.; at 32 mins. about 90 per cent of plaster had fallen below bracket course; plaster still on bracket on all four sides and down to 9 ft. up on west. 28 mins. to 1 hr. 10 mins.—some spalling of outer shells and very slight amount of cracking, mostly at corners; at end of this period northeast corner was spalled 5 to 6 ft. up, also on west 4 to 5 ft. up, and on east 6 to 7 ft. up; shell loose on north 7 to 8 ft. up. 1 hr. 32 mins.—several corners spalled in upper

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two thirds of column. 1 hr. 41 mins.—outer shell spalled on north 8th course; plaster on at bracket, except on north side. 2 hrs. 30 mins.—little change during past 50 mins., except for a few spalls of outer shells in upper half. 3 hrs. 30 mins.—no change except outer shell spalled on west, 9th course. 3 hrs. 45 mins.—maximum expansion. 4 hrs.—little change, except for a few vertical cracks, $\frac{1}{16}$ to $\frac{1}{8}$ in., near the top; plaster still on parts of bracket. 4 hrs. 18 mins.—outer shell spalled on east, 9th course; no inner shells spalled before failure; no spalling of outer shells on four lower courses; deflections not measured. 4 hrs. 25½ mins.—failure with local buckling about 9½ ft. above base.

After failure.—Shale tile almost all off; could not be examined; Ohio semifire-clay tile quite shattered, no bond with mortar or fill; semifire clay, New Jersey district, tile in fairly good condition except for a number of fine cracks; fair bond with fill. Concrete fill full, no voids. Mortar joints on sides were fairly full. (Figs. 78, 138, and 46.)

TEST NO. 77. PLATE AND ANGLE. 4-IN. HOLLOW CLAY TILE COVERED WITH ⅝ IN. LAYER OF LIME PLASTER; LIMESTONE-CONCRETE FILL. UPPER 4 COURSES, SEMIFIRE CLAY, NEW JERSEY DISTRICT; MIDDLE 4 COURSES, SURFACE CLAY, CHICAGO DISTRICT; LOWER 4 COURSES, SURFACE CLAY, BOSTON DISTRICT.—½ min.—most of plaster spalled off, exposing nearly three-fourths of total tile surface. 15 mins.—practically all of plaster down, except small amount in places near top; some outer shells of tile beginning to buckle in middle courses. 20 to 50 mins.—cracking and some spalling of outer shells, mostly at corners, in middle 4 courses; a few cracks in lower 4 courses, less than $\frac{1}{16}$ in. wide. 1 hr. 10 mins.—very little change during 20 mins., though cracks opened slightly; no cracks observed in upper 4 courses. 1 hr. 35 mins.—a few fine cracks in upper 4 courses, not over $\frac{1}{32}$ in. 1 hr. 49 mins.—a number of horizontal cracks in lower 4 courses up to ⅛ in. wide; outer shell spalled on west 3 to 4 ft. up. 2 hrs. 20 mins.—horizontal and vertical cracks in lower 4 courses opening up ⅛ to $\frac{1}{16}$ in.; cracks in upper 4 courses not opening up. 3 hrs. 22 mins.—a few tile in lower courses curved out, splitting near ends; not much change in middle 4 courses. 4 hrs.—cracks in lower 4 courses open $\frac{1}{16}$ to ½ in.; northeast corner spalled 2 to 3 ft. up; possibly fusion 3 ft. up. 4 hrs. 10 mins.—maximum expansion. 4 hrs. 17 mins.—more spalling of outer shells in middle courses; no inner shells spalled. 4 hrs. 20 mins.—tile bulged out from steel just above middle on north and south sides; very decided fusion 1 to 3 ft. up. 4 hrs. 26 mins.—cracks in lower 4 courses open $\frac{1}{16}$ to ½ in.; outer shell 5th to 7th course, north side, spalled; cracks in upper 4 courses open not over $\frac{1}{32}$ in. 4 hrs. 26 mins. to 4 hrs. 30 mins.—considerable spalling of outer shells in middle 4 courses; all inner shells, all courses, in place before failure. Deflections not measured. 4 hrs. 42¼ mins.—failure with buckling to west, maximum at 6 ft. above base.

After failure.—All of upper four courses and middle four courses of tile down, examination not possible. Lower four courses much shattered by vertical and horizontal cracks; incipient fusion through outer shells and through webs where exposed; pieces of this tile bent, curled, and discolored. Concrete fills reentrant portions fully. Appearance of steel indicated that concrete had not fully filled space between flanges and tile. (Figs. 78, 139, and 46.)

(*f*) GYPSUM BLOCK PROTECTIONS

TEST NO. 64. ROLLED H. 4-IN. WESTERN GYPSUM BLOCK (SOLID). HOLLOW GYPSUM-BLOCK FILL.—10 mins.—fine surface checks developing, about $\frac{1}{32}$ in. wide. 30 mins.—fine surface checks all over faces, about ½ in. apart both ways. 48 mins.—corners spalled not over ½ in. by ½ in. 1 hr.—very little change. 1 hr. 30 mins.—surface checks about ⅛ in. wide by ⅛ in. deep; a few vertical joints opened not over

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$\frac{1}{32}$ in. 2 hrs. 30 mins.—both horizontal and vertical joints opened about $\frac{1}{8}$ in.; mortar joints project $\frac{1}{4}$ in. 2 hrs. 40 mins.—block fell from bracket on north, exposing west part of bracket steel. 3 hrs. 10 mins. to 3 hrs. 25 mins.—spalling on southwest and northeast corners, just below middle, to 2 in. back. 3 hrs. 35 mins.—surface checks $\frac{1}{8}$ in. wide by $\frac{1}{2}$ in. deep; all cracks open to about $\frac{1}{4}$ in. 3 hrs. 45 mins.—mortar joints throughout project about $\frac{1}{2}$ in. 3 hrs. 55 mins.—piece of block down on southwest side of bracket; blocks on northeast and southeast sides of brackets stand out. 4 hrs. 5 mins.—block down on northeast side of bracket, exposing rest of bracket steel. 4 hrs. 10 mins.—mortar joints project $\frac{3}{4}$ in.; tile appears to be about 3 in. thick. 4 hrs. 15 mins.—block down on southeast side of bracket; both sides of south bracket now exposed; bracket steel glowing dull red. 4 hrs. 20 mins.—blocks on east and west faces tilting out from steel at 7 ft. up; surface checks appear to be about 1 in. deep. 4 hrs. 30 mins.—nearly all of block down on west 6 to 7 ft. up; all fell on east, from 4 ft. up to top of column, on south from 1 to 6 ft. up, exposing flange for 5 ft., also down on north, 10 to 11 ft. up. 4 hrs. 32 mins.—maximum expansion. 4 hrs. 39 mins.—all blocks down on west from 1 ft. up to top. 4 hrs. 30 mins.—slight deflection to northeast noted, increasing to $\frac{3}{8}$ in. at 4 hrs. 41 mins. 4 hrs. 43$\frac{1}{4}$ mins.—failure with buckling to east, maximum at 5 ft. above base.

After failure.—Exposed column flange on south greatly oxidized during test. Gypsum blocks shrunken to thickness of 3$\frac{1}{4}$ to 3$\frac{1}{2}$ in. Surface of blocks checked into $\frac{1}{4}$-in. squares, cracks extending inward to depth of 1$\frac{1}{4}$ to 2 in. from surface on sides and to 3$\frac{1}{2}$ at corners, the corner cracks extending in diagonally. Gypsum in this zone is brownish green. Further in gypsum is much more crumbly than new material, showing that the heat had affected it. (Figs. 74 and 127.)

TEST NO. 65. PLATE AND CHANNEL. 2-IN. WESTERN GYPSUM BLOCK (SOLID); SOLID GYPSUM-BLOCK FILL.—11 mins.—fine surface checks beginning to develop. 23 mins.—a few fine cracks and slight corner spalls; surface checks more distinct, forming in $\frac{1}{2}$ in. squares. 29 mins.—a few vertical joints open $\frac{1}{16}$ in. 48 mins.—surface checks $\frac{1}{16}$ in. wide by $\frac{1}{4}$ in. deep, distinct throughout courses; cracks $\frac{1}{2}$ in. deep; some corners spalled $\frac{1}{2}$ in. back, corners generally ragged. 1 hr. 5 mins.—cracks in all joints about $\frac{1}{8}$ in. wide; surface checks $\frac{1}{8}$ in. wide. 1 hr. 30 mins.—shrinking of tile apparent, mortar joints project $\frac{1}{16}$ to $\frac{1}{8}$ in.; cracks in joints now $\frac{1}{8}$ in. wide. 1 hr. 45 mins.—block fell on west 4 to 5 ft. up, exposing edges of flanges. 1 hr. 49 mins.—block fell on south 5 to 6 ft. up; mortar in joints projects out generally, $\frac{1}{4}$ to $\frac{3}{8}$ in. 2 hrs.—blocks fell on north 2 by 6 ft. up, steel still covered by mortar. 2 hrs. 5 mins.—blocks fell on east 2 to 6 ft. up, on west 2 to 5 ft. up; three top courses still intact. 2 hrs. 9 mins.—blocks down on south and west 5 to 6 ft. up. 2 hrs. 10 mins.—edges of flanges exposed where blocks have fallen, but steel plates are covered with about 1 in. of mortar in all cases. 2 hrs. 12 mins.—block down on west 6 to 7 feet up; piece fallen on northwest corner at bracket. 2 hrs. 17 mins.—mortar in joints projects $\frac{3}{8}$ in. 2 hrs. 20 mins.—maximum expansion; mortar fallen on north exposing steel; blocks down on north and west 7 to 8 ft. up. 2 hrs. 11 mins.—center deflection less than $\frac{1}{8}$ in. 2 hrs. 21$\frac{1}{2}$ mins.—failure with local buckling about 3 ft. above base.

After failure.—Blocks shrunken to thickness of 1$\frac{1}{2}$ to 1$\frac{3}{4}$ in. Surfaces of blocks checked into $\frac{1}{4}$ to $\frac{1}{2}$ in. squares, cracks being $\frac{1}{16}$ to $\frac{1}{8}$ in. wide at surface and $\frac{1}{2}$ to $\frac{3}{4}$ in. deep on sides; cracks at corners extending in diagonally 1 to 1$\frac{1}{4}$ in. Outer dehydrated surface quite firm and greenish in color in places; hardness extends to inner zone of checking where color changes from white to brownish. Gypsum farther in is softer than material not exposed to fire. Galvanizing on corrugated sheet-metal ties placed in the joints was removed; otherwise they were not oxidized or corroded. (Figs. 74 and 128.)

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TEST NO. 66. LATTICED CHANNEL. 2-IN. EASTERN GYPSUM BLOCK (SOLID); POURED GYPSUM FILLING.—15 mins.—fine surface checks beginning to develop. 30 to 40 mins.—surface checks more distinct, forming in ¼ to 1 in. squares, mostly in lower half. 1 hr. 15 mins.—joints open to $\frac{1}{16}$ in. increasing to ⅛ in. at 1 hr. 35 mins. 1 hr. 35 mins.—mortar in joints project $\frac{1}{16}$ in. 2 hrs.—surface checks now cover whole surface, and are ¼ in. to ½ in. in depth; joints open to ⅜ in. maximum. 2 hrs. 5 mins.—block on east 5½ ft. up, tilting out ¾ in. at top. 2 hrs. 14 mins.—small spall on southwest corner at bracket, block below bracket on west tilting out at top. 2 hrs. 15 mins.—block down on east in 4th course, exposing lattice bars where unprotected by filling. 2 hrs. 22 mins.—block fell on east 8th course, exposing steel. 2 hrs. 26 mins.—blocks fell on north 1½ to 5½ ft. up, exposing steel. 2 hrs. 30 mins.—block fell on south, 4th course up, exposing steel. 2 hrs. 32 mins.—maximum expansion; blocks fell on south and west 2d course up. 2 hrs. 33 mins.—center deflection less than ⅛ in. 2 hrs. 36 mins.—failure with local buckling about 3½ ft. above base, causing column to deflect to west.

After failure.—Filling fairly full, but has some pockets, 2 in. deep; filling extends out between lattice bars, but not much beyond; very crumbly where exposed. Blocks shrunken to thickness of 1⅞ ins.; little change in color. Surface checks are $\frac{1}{16}$ in. wide at surface and ½ in. to ¾ in. deep. Strips of wire mesh in joints not greatly oxidized. (Figs. 74 and 128.)

TEST NO. 67. ROLLED H. 4-IN. EASTERN GYPSUM BLOCK (SOLID); POURED GYPSUM FILLING.—17 mins.—fine surface checks, forming in 1-in. squares beginning to show. 34 mins.—surface checks more marked, $\frac{1}{32}$ to $\frac{1}{16}$ in. wide; checking most pronounced in lower half. 1 hr. 5 mins.—surface checks ¼ in. deep and ⅛ to ¼ in. wide; corner cracks ½ to 1 in. deep and 1 to 2 in. long. 1 hr. 20 mins.—joints beginning to open up; several horizontal cracks 1 in. deep across faces; surface checks now forming in ½ to ¾ in. squares. 1 hr. 50 mins.—very little change; surface checks ¼ to ½ in. deep are general. 2 hrs. 10 mins.—a few mortar joints project $\frac{1}{16}$ to ⅛ in. 2 hrs. 21 mins.—pieces of corners at bracket spalled. 2 hrs. 40 mins.—joints open to ⅛ in.; surface checks ¾ in. deep and ¼ in. wide. 3 hrs. 45 mins.—mortar in joints projects ¼ in., vertical joints open to ¼ in., little change in surface checks. 4 hrs. 40 mins.—mortar projects to ½ in.; otherwise little change. 4 hrs. 52 mins.—top of block tilts out 1 in. on north 8½ ft. up. 4 hrs. 54 mins.—blocks fell on south 3d and 4th courses; mortar still covers steel quite generally; piece of block 4 in. wide also down at southwest corner in 5th course; joints near top open $\frac{1}{16}$ in. 5 hrs. 1 min.—maximum expansion. 5 hrs. 5 mins.—blocks next to those fallen tilt away from steel; steel now exposed on south 3 to 4 ft. up. 5 hrs. 7 mins.—blocks down on west 3d to 6th course, exposing filling and flange edges. 5 hrs. 13 mins.—block fell on west 2d course; west edge of flange exposed 4 to 6 ft. up; little change after this until failure. 5 hrs. 31½ mins.—failure with local buckling about 4 ft. above base.

After failure.—Fill fairly full, but has some pockets 2 in. deep; mortar where still in place, covers flanges ¾ to 1 in.; steel fluxed for 30 in.-length near point of failure; edges of flanges attacked where exposed, unaffected where not exposed; strips of wire mesh oxidized through where they had been exposed to the fire; blocks shrunk to thickness of 3⅝ to 3⅞ ins. Surface check cracks formed in ¼ to ½ in .squares, cracks $\frac{1}{16}$ to $\frac{1}{16}$ in. wide at surface and generally 2 in. deep, although some extend completely through blocks; radial cracks at corners 2¼ in. deep. Block colored brownish yellow in outer 1 in., further in stained dark from oxidation of wood fiber. (Figs. 75 and 129.)

TEST NO. 67 A. ROLLED H. 4-IN. EASTERN GYPSUM BLOCK (SOLID); POURED GYPSUM FILLING.—15 mins.—fine surface checks beginning to show; fine cracks on ends of blocks. 45 mins.—surface checks forming in ¼ to 1 in. squares, most pronounced on west. 1 hr. 30 mins.—vertical joints opening slightly. 2 hrs.—cracks in both vertical

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and horizontal joints opening up perceptibly; surface checks ¼ in. deep. 3 hrs.—vertical joints near middle of column open to ¼ in. 3 hrs. 30 mins.—all joints open from ⅛ to ¼ in.; mortar joints project ¼ in.; minor spalling from corners. 4 hrs.—all blocks have shrunk considerably; joints open ⅛ to ¼ in. 4 hrs. 30 mins.—small pieces spalling from corners; larger piece fallen from northwest corner, 7 ft. up. 5 hrs. 15 mins.—edge of bracket steel exposed on south side. 5 hrs. 15 mins. to 5 hrs. 45 mins.—considerable spalling at corners. 5 hrs. 45 mins.—maximum expansion. 5 hrs. 56 mins.—blocks down on north 2d and 3d courses, and on west 2d course; steel exposed. 6 hrs. 14 mins.—blocks now down on north from bottom to 5½ ft. up. 6 hrs. 20 mins.—blocks down on east 2d course and on south 3d course. 6 hrs. 21 mins.—block down on west 7th course. 6 hrs. 10 mins.—slight deflection to southwest increasing to ¼ in. at 6 hrs, 16 mins. 6 hrs. 24½ mins.—failure with buckling to south, maximum at 4¼ ft. above base.

After failure.—Filling fairly full. Column flange on north greatly oxidized for 3 ft. near point of failure. Blocks shrunk to thickness of 3½ to 3⅝ in.; gypsum very crumbly. Surface checks ¼ in. wide, extend in to depth of 2¾ in. (Figs. 75 and 130.)

(*g*) BRICK PROTECTIONS

TEST NO. 68. ROLLED H. 2¼-IN. CHICAGO COMMON BRICK LAID ON EDGE; BRICK FILL.—13 mins.—a number of $\frac{1}{16}$-in. cracks in joints. 15 mins.—some cracking and spalling at corners. 16 to 18 mins.—several vertical cracks developing in middle sections, one on north face ⅜ in. wide, 2 ft. long. 23 mins.—brick fell on east from 3½ to 8½ ft. up, exposing north flange to depths of from 1 to 3 in. back from edge. 26 mins.—brick fell on west, 4 to 8½ ft. up, exposing south flange for width of 1 in., bricks bulged out ¼ in. from flanges on north and south in this region, increasing to ½ in. at 36 mins. 36 mins.—brick in upper third cracked somewhat; brick almost intact up to 3½ ft. above base. 1 hr. 4 mins.—very little change, bulging in middle slightly increased. 1 hr. 10 mins.—maximum expansion. 1 hr. 20 mins.—slight deflection to southeast noted, increasing to ¼ in. at 1 hr. 37 mins. 1 hr. 40¾ mins.—failure with local buckling 6½ to 8 ft. above base.

After failure.—Brick soft and crumbly and cracks readily. Mortar joints apparently quite full. (Figs. 76, 118, and 166.)

TEST NO. 69. ROLLED H. 3¾-IN. CHICAGO COMMON BRICK LAID FLAT. BRICK FILL.—

First Test.—Trouble with one gas burner developed soon after start of test and gas was shut off at 37¼ mins. Test postponed for two days. Column observed after test was discontinued. One-half to two-thirds of brick cracked through vertically in one or more places, cracks from very fine to $\frac{1}{16}$ in. wide. Slight flaking and spalling of corners noted. Temperature of steel at end of test 45° C, attaining a maximum of 140° C, 3 hrs. 20 mins. later.

Second Test.—1 hr. 30 mins.—slight flaking, no spalling; cracks developed in first test not opening up. 3 hrs. 30 mins.—no cracks over $\frac{1}{16}$ in., brick flaking off at corners. 3 hrs. 47 mins.—cracks are vertical and generally very fine, maximum $\frac{1}{16}$ in. and not more than 2 ft. in length. 4 hrs. 16 mins.—very little change, cracks open to ⅛ in. maximum; no fusion. 4 hrs. 52 mins.—decided fusion in lower 3 ft. only. 5 hrs. 20 mins.—maximum expansion. 5 hrs. 38 mins.—fusion extended up to 6 ft., fused brick run down to base at corners. 6 hrs. 10 mins.—fusion up to 10 ft. above base. 6 hrs. 27 mins.—fusion extends up to 11 ft. above base; no spalling, except surface flaking, occurred before failure. 6 hrs.—slight center deflection to west noted, increasing to ⅝ in. at 7 hrs. 11 min. 7 hrs. 13¼ mins.—failure, with buckling to west, maximum at 5¼ ft. above base.

After failure.—Fusion at bottom fluxed away about ½ in. of brick. Brick at bracket had just begun to fuse. (Figs. 76, 131, and 168.)

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(b) REINFORCED-CONCRETE COLUMNS

TEST NO. 70. SQUARE VERTICALLY REINFORCED; LIMESTONE CONCRETE.—16 mins.—corners of column glowing slightly in lower 3 ft. 34 mins.—column luminous entire length; slight flaking on corners near bottom. 1 hr. 11 mins.—fine vertical crack on east, 2 to 3 ft. up. 1 hr. 28 mins.—similar crack 6 in. long 1 ft. from base. 3 hrs. 2 mins.—similar crack on east, 12 in. long near middle of column. 5 hrs.—maximum expansion. 6 hrs. 39 mins.—fine vertical cracks with ends running horizontally to corners appeared on various faces. 7 hrs. 50 mins.—cracks on east opening slightly; also a few additional fine cracks. 8 hrs.—column still supporting working load with no apparent change; no spalling. 8 hrs. 1 min.—load increased with fire going until failure occurred under 294 000 lbs. about 11 ft. above base, at 8 hrs. 40¼ mins.

After failure.—Concrete near outside calcined, but had hard surface due to partial fusion of the sand in the concrete. (Figs. 76 and 132.)

TEST NO. 71. SQUARE VERTICALLY REINFORCED; TRAP CONCRETE.—20 mins.—column glowing entire length. 2 hrs. 40 mins.—maximum expansion. 3 hrs.—fine vertical cracks on east and west near bottom. 3 hrs. 10 mins.—slight flaking at corners near middle. 4 hrs. 10 mins.—the fine cracks on east and west faces extending in length. 4 hrs. 40 mins.—several additional fine cracks on east and west faces, 4 to 24 in. long. Very little change before failure; no spalling. 5 hrs.—slight deflection to north noted, increasing to ¼ in. at 7 hrs. 22 mins. 7 hrs. 22¾ mins.—failure by compression about 5 ft. above base, reinforcing bars buckling outward.

After failure.—Concrete fused to average depth of 1 in. up to 7½ ft. above base; incipient fusion from 7½ to 9 ft. up, no fusion above. Concrete dry and loose in texture at top. Bars apparently straight, except where buckled at point of failure. Concrete cracked at bars at some points, but in general its condition at the corners was about the same as at the middle of the sides. (Figs. 77, 133, and 169.)

TEST NO. 72. ROUND VERTICALLY REINFORCED; LIMESTONE CONCRETE.—20 mins.—slight surface flaking. 1 hr.—column glowing entire length. 5 hrs.—maximum expansion. 5 hrs. 26 mins.—no spalling or cracking noted. 7 hrs. 55 mins.—a few cracks noted on east and west faces ⅛ to ⅛ in. wide and 3 to 10 in. long, as follows: On east at 3 and 8 ft. up, on west at 2 and 6 ft. up, and at bracket. 8 hrs.—column still supporting working load with no apparent change; no spalling and but few cracks, as noted; deflection less than ⅛ in. 8 hrs. 2 mins.—load increased with fire going until failure occurred under 250 000 lbs., about 6 ft. above base, at 8 hrs. 4½ mins., reinforcing rods buckling outward.

After failure.—Surface hard immediately after test due to partial fusion of sand. A few days after test concrete flaked off to depth of 1 in. due to calcination of limestone. (Figs. 77, 134, and 170.)

TEST NO. 73. ROUND VERTICALLY REINFORCED. TRAP CONCRETE.—19 mins.—piece of concrete about 8 in. wide and ½ in. deep spalled on west 2 ft. above base. No cracking or other spalling noted before failure. Furnace gases very heavy, making observation difficult; deflection not measured. 4 hrs. 10 mins.—maximum expansion. 7 hrs. 57½ mins.—failure at 2 to 4 ft. up, concrete crushing and shearing on inclined planes; reinforcing bars buckled outward.

After failure.—Concrete fused to depth of 1 in. at break, fusion being more decided in lower than in upper half. Little or no concrete had run. A large number of fine vertical and horizontal cracks present more or less over whole surface of column. (Figs. 77 and 135.)

TEST NO. 74. HOOPED REINFORCED; LIMESTONE CONCRETE.—40 mins.—no spalling or cracking noted. 53 mins.—two fine cracks 3 in. long, on west, 5½ ft. up, opening to ⅛ in. wide and 10 in. long at 2 hrs. 36 mins. 5 hrs. 50 mins.—maximum expan-

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sion. 6 hrs. 50 mins.—several cracks ⅛ in. wide and about 8 in. long at 2½ ft. up, and one crack $\frac{1}{16}$ in. wide and 8 in. long on east, 5½ ft. up; no cracks above. 8 hrs.—column still supporting working load with no apparent change; no spalling; deflection less than ⅛ in. 8 hrs. 5 mins.—load increased with fire going until failure occurred under 243 000 lbs. about 3 ft. above base, at 8 hrs. 6½ mins.

After failure.—Eight breaks in spiral reinforcement occurred near failure point. Vertical bars buckled out 3 in. Concrete spalled outside of spiral 2 to 4 ft. up; otherwise no plane of cleavage at spiral. (Figs. 77, 136, and 171.)

TEST NO. 75. HOOPED REINFORCED. TRAP CONCRETE.—

First Test.—Gas shut off after 30 mins. to repair burner and test postponed till next day; column apparently not affected.

Second Test.—36 mins.—column glowing dull red for full length. 40 mins.—fine vertical crack, 4 in. long, on southeast 5½ ft. up, increasing to 12 in. long at 1 hr. 45 mins. 1 to 2 hrs.—some six or eight fine cracks, 2 to 12 in. long noted in lower half. 2 hrs. 20 mins.—all cracks opening slightly. 3 hrs.—many very fine cracks on all surfaces. 4 hrs. 50 mins.—maximum expansion. 6 hrs. 25 mins.—cracks opened somewhat; observation difficult. 8 hrs.—column still supporting working load with little apparent change; no spalling; deflection at 8 hrs. less than ⅛ in. 8 hrs. 1½ mins.—load increased with fire going, failure occurring under load of 163 000 lbs., about 3 ft. above base, at 8 hrs. 1¾ mins.

After failure.—Concrete fused to a depth of 1½ in. up to 11 ft. above base; concrete fluxed off to depth of 1 in., 7 to 9 ft. above base. No fusion in upper 1½ ft. due to excess of mortar near surface. Failure apparently due to yielding of spiral, although no break in it occurred. Vertical bars buckled out 1 in. at failure point. Concrete outside of spiral shells off readily. (Figs. 77 and 137.)

NOTE.—For tests Nos. 76 and 77, see under paragraph (*e*) above, hollow clay-tile protections, after test No. 63.

(*i*) TIMBER COLUMNS

TEST NO. 78. LONG-LEAF PINE WITH CAST-IRON CAP AND PINTLE; PROTECTED BY A 1-IN. LAYER OF PORTLAND-CEMENT PLASTER ON METAL LATH.—5 to 8 mins.—vertical cracks in lower half near northwest and southeast corners open to ⅜ in. and 3 to 5 ft. long. 9 mins.—vertical crack at northeast corner at bracket $\frac{1}{16}$ in. wide, 12 in. long. 11 to 40 mins.—finish coat bulging out and spalling in lower half on north and south sides. 16 mins.—crack at bracket on west, ⅛ by 8 in. 20 mins.—crack at pintle, northeast corner, open ¼ in. 44 mins.—⅛ in. vertical crack near northeast corner 1½ to 6½ ft. up. 52 mins.—lath and supporting channel buckle out 1½ in. on south side at west corner, 4 ft. up. 55 mins.—cracks at bracket on northwest corner open ⅛ in., on southwest corner ¼ in. 57 mins.—spurts of flame issue at southwest corner where lath buckled out. 58 mins.—crack at bracket on northeast corner open ⅜ in., on southeast open $\frac{1}{16}$ in. 1 hr. 13 mins.—flames issue from cracks at bracket on west; flames free and full at southwest corner, 4 ft. up. 1 hr. 21 mins.—plaster spalled to lath, 8 by 12 in., on west, 2½ ft. up; flames issue at this point. 1 hr. 24 mins.—flames issue from crack at bracket on northeast corner. 1 hr. 57 mins.—head of column going down quite rapidly; column cap apparently level; crack noted in plaster at base of cap. 1 hr. 58 mins.—plaster spalled to lath, 12 by 12 in. on east, 2 ft. up. 2 hrs. 6 mins.— cap still level. 2 hrs. 8 mins.—little change in plaster during past 50 mins., except as noted. 2 hrs. 9 mins.—small piece of plaster spalled on northwest corner, at lower part of bracket. Settlement of top of column due to heating of cap $\frac{1}{16}$ in. at 1 hr. 10 mins., ¼ in. at 1 hr. 50 mins., 1¼ in. at 2 hrs., and 3¼ in. at 2 hrs. 15 mins. (See Fig. 47.) 2 hrs. 15¼ min.— failure, due to cracking of cap. 2 hrs. 17 mins.—water applied to column extinguishing flames at 2 hrs. 21 mins.

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After failure.—Cap broken into three pieces by two transverse cracks across middle portion. A 2 by 2 by ½ in. chip broke off bottom edge of pintle on north. Beam ends charred to depth of 1 in. on sides and ends; inner surface surrounding pintle scarcely charred. Sides of column charred to depth of 1¼ in. in lower two-thirds and to about 1 in. toward top, minimum section of unburned wood, about 8 by 8 in.; vertical cracks on sides extend inward not over ½ in. Top bearing surface of column not charred except for ¼ in. at edges, but was crushed and frayed, the fibers being bent out over 3 in. beyond sound wood at the sides, and pushed up 2 to 3 in. into cracks in cap. A piece was sawed from southwest corner of top and fibers were found to have been crushed and turned over for a depth of 1¾ in. below surface. Wood crushed down on northwest corner 2 in. more than on south side of bearing surface, due to pintle bearing on this side and transmitting the blow from ram at failure Length of column below cap before test, 11 ft. 1¾ in.; average after test, 10 ft. 9 in.; decrease in length, 4¾ in. (Figs. 79, 140, and 47.)

TEST NO. 79. LONG-LEAF PINE WITH CAST-IRON CAP AND PINTLE; UNPROTECTED.—2 mins.—surface of column blazing in lower 3 ft. 3 mins.—fine horizontal checks due to charring appeared. 9 mins.—column flaming all over. 11 mins.—three or four vertical cracks beginning to show near middle of each side, $\frac{3}{16}$ in. wide. 18 to 23 mins.—horizontal cracks showing in beam ends at top increasing to ½ in. at 32 mins. 27 mins.—horizontal checks $\frac{3}{16}$ in. wide and 1½ in. apart quite general. 30 mins.—lazy reddish flames from combustion of wood envelop whole column. 32 mins.—vertical cracks in charred column generally ⅜ in. wide. 42 mins.—head of column going down fast, but no visible sign of distress. 44 mins.—vertical cracks ⅝ in. wide; horizontal checks ¼ in. wide. Settlement of head of column due to heating of cap $\frac{1}{16}$ in. at 25 mins., 1⅜ in. at 40 mins., and $3\frac{1}{16}$ in. at 46 mins. (See Fig. 47.) 50 mins.—failure due to cracking of cap.

After failure.—Cap broken into two pieces, cracking transversely at center. Pintle intact except for slight rounding of lower bearing surface. Beam ends charred to depth of 1½ in. on sides and ends; inner surface surrounding pintle barely scorched. Sides of column charred to depth of 1¼ in.; minimum section of unburned wood 8¾ by 8¾ in. Vertical cracks on sides extend inward to a depth of ¾ in. below sound wood. Column split in two in upper 3 ft. due to end of pintle being forced down through cap into top of column at failure. Top bearing surface very little charred but fibers crushed and broomed, forced outward 3 in. beyond sides, also down into crack in column and up into crack in cap. Fibers tough and ropy and bent over for a length of 3 in. Length of column before test, 11 ft. $1\frac{7}{16}$ in.; average after test, 10 ft. $7\frac{7}{16}$ in.; decrease in length, 6 in. (Figs. 80, 140, and 47.)

TEST NO. 80. LONG-LEAF PINE, WITH STEEL-PLATE CAP AND TIMBER-STRUT BEARING; COLUMN AND CAP PROTECTED BY ONE THICKNESS OF ⅜-IN. GYPSUM WALL BOARD.—1 min.—kalsomine burned off, showing filler in joints and nailing. 5 mins.—paper on outer surface of wall board charring and flaking, about one-half off at 7 mins. 15 mins.—about two-thirds of outer paper now off; no curling of wall board. 16 to 18 mins.—cracks noted near corners at bracket on east and west sides, up to ⅛ in wide, extending and increasing to ¼ in. on west at 26 mins. 19 to 25 mins.—horizontal cracks developed on north, south, and west faces from 1 to 4 ft. up; wood burning freely at cracks, also a little along corners. 27 mins.—corner beading buckled out at several places, corners burning freely up to 7 ft. above base and to top at 40 mins. 31 mins.—cracks and free burning of wood on east face to 9½ ft. up. 32 mins.—about two-thirds of wall board on west side of cap fell, exposing steel. 35 mins.—board fell at cap on east, exposing all of steel. 37 mins.—board in lower half buckled out at corners; horizontal cracks about 18 in. on centers on all sides with free burning of wood. 41

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to 54 mins.—board fell off in places, exposing wood; at end of this period wood was exposed on north at 4 ft. up, on east 2 to 8 ft. up, and on south 2 to 10 ft. up; board remaining in place, much shattered. 1 hr. 3 mins.—continued cracking and falling of board; steel cap slanting slightly to north. 1 hr. 6 mins.—all board down on north 2 to 4 ft. and 7 to 8 ft. up; on west from 4 ft. up to top. 1 hr. 10 mins.—cap tilted, south edge dropped 1½ in. Settlement of top of column due to heating of cap $\frac{1}{16}$ in. at 15 mins., ½ in. at 40 mins., and 2⅛ in. at 1 hr. 10 mins. 1 hr. 13 mins.—failure by tilting of cap, and slipping of same on top bearing strut, column and cap being carried to north.

After failure.—Side plates of cap bent, also inner bolt on south side; bearing plate dished up 1 in. at middle. Strut did not slip and its upper bearing surface was uninjured; sides of strut charred to depth of ½ in., lower bearing surface of strut charred about ¼ in. on edges, brown over rest of area. Fibers crushed and bent over 1½ to 3 in. Strut cracked by longitudinal shearing. Sides of column charred to depth of 1 to 1¼ in.; minimum section of unburned wood, 9⅜ by 8¾ in.; vertical cracks burnt in ¾ to 1 in. deeper. Top bearing surface of column charred only at edges; surface convex, middle being about ⅜ in. higher than edge; fibers crushed and bent over to south for depth of about ½ in.; bearing surface smooth and hard. Length before test, 12 ft. 2 in.; average after test, 12 ft. 1 in.; decrease in length, 1 in. (Figs. 81, 141, and 47.)

Test No. 81. Long-leaf Pine with Steel-Plate Cap and Timber-Strut Bearing; unprotected.—2 mins.—surface of column blazing, lower 3 ft. 6 mins.—columns burning freely on south and west, lower half. 5 mins.—a few vertical cracks near middle. 8 mins.—column charred; irregular surface checks $\frac{1}{16}$-in. wide, quite general. 16 mins.—several vertical cracks ¼ in. wide on all sides; vertical cracks, $\frac{1}{16}$ in. wide. 19 mins.—gases heavy, making observation difficult. 32 mins.—color noted in cap. 34 mins.—all cracks about ⅛ in. wide; settlement of top of column due to heating of cap $\frac{1}{16}$ in. at 20 mins., increasing to $\frac{1}{16}$ in. at 30 mins., and 1$\frac{1}{16}$ in. at 34 mins. 35 mins.—failure with top of column sliding to south and west; fire kept burning until 43½ mins., as it was not possible to ascertain definitely if failure had taken place, owing to heavy smoke; column yielded quite gradually, only slight report heard at failure. 45 mins.—water applied to column extinguishing flames completely at 55 mins.

After failure.—Upper part of side plates of cap bent toward the east; bearing plate dished up ¾ in. at middle. Strut did not slip and its upper bearing surface was uninjured; lower bearing surface of strut charred to depth of 1/16 in. and fibers crushed and bent over to depth of ½ to 1 in.; strut split by longitudinal shearing. Sides of column charred to depth of 1$\frac{1}{16}$ in.; minimum section of unburned wood 9 by 9 in. Top bearing surface of column charred not deeper than 1/16 in. except at edges; fibers crushed and bent over to north for a depth of about ½ in. Length before test, 12 ft. 2 in.; average after test, 12 ft. 1⅛ in.; decrease in length, ⅞ in. (Figs. 81, 141, and 47).

Test No. 82. Douglas Fir with Cast-Iron Cap and Pintle; unprotected.—3 mins.—surface of column blazing in lower 3 ft.; horizontal surface checks due to charring, 1½ in. on centers. 10 mins.—column blazing freely all over; several vertical checks on all sides, $\frac{1}{16}$ in. wide, small pieces of charcoal falling. 14 mins.—horizontal checks $\frac{1}{16}$ in. wide. 24 mins.—horizontal and vertical checks about ¼ in. wide. 25 mins.—column flaming all over, but not so freely as pine columns at this stage. 36 mins.—pieces of charcoal falling off under cap, indicating rapid dropping of head of column. 38 mins.—cap nearly level; no color. 41 mins.—buckling under cap more pronounced; noise heard at bearing. 42½ mins.—cap still nearly level. 44 mins.—wood fibers under cap appear to crush and buckle out. Settlement of top of column due to heating of cap $\frac{1}{16}$ in. at 15 mins., 1 in. at 30 mins., 3½ in. at 41 mins., and 5⅛ in. at 45 mins. 45¼ mins.—failure due to cracking of cap.

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After failure.—Cap broken transversely into two pieces. Small pieces chipped from outer edge of pintle on southwest at bottom; otherwise pintle uninjured. Beam ends charred on sides and ends to maximum depth of 1 in.; inner surface of wood surrounding pintle very slightly charred in places. Sides of column charred to depth of 1 1/8 in.; minimum section of unburned wood, 9 1/2 by 9 3/4 in.; vertical cracks did not extend into uncharred wood. Top bearing surface of column charred not more than 1/8 in. deep, but was broomed and crushed; fibers bent out beyond sides of column, also pushed up 3 in. into crack in cap. A piece cut out of southwest corner showed fibers crushed and turned over 3 to 3 1/4 in. below bearing surface; fibers tough, hardly scorched. Length before test, 11 ft. 1 5/8 in.; average after test, 10 ft. 6 in.; decrease in length, 7 5/8 in. (Figs. 82, 140, and 47.)

TEST NO. 83. DOUGLAS FIR, WITH STEEL-PLATE CAP AND TIMBER-STRUT BEARING; UNPROTECTED.—2 1/2 mins.—surface of column blazing lower half; top just beginning to burn. 2 to 4 mins.—crackling noises heard. 5 mins.—column blazing all over. 15 mins.—very little blazing, hardly any crackling, secondary air gate found shut; opened. 18 mins.—horizontal surface checks 1/8 in. wide, 1 in. on centers; 2 to 4 vertical cracks on each side not over 1/8 in. wide. 24 mins.—column again burning freely. 31 mins.—horizontal checks 1/4 in. wide. 37 mins.—vertical cracks up to 1/8 in. wide; very little crackling. 38 mins.—cap tilted, south edge dropped 1 3/8 in.; settlement of top of column, due to heating of cap 1/8 in. at 14 mins., 1 1/8 in. at 30 mins., and 2 1/8 in. at 38 mins. 38 1/2 mins.—failure with top of column sliding to north carrying cap with it. 40 1/2 mins.—water applied to column extinguishing flames completely at 45 1/2 mins.

After failure.—Side plates of cap bent outward at top, also inner bolt on north bent out; bearing plate dished up 1 in. at middle in longitudinal direction. Strut did not slip, and its upper bearing surface was uninjured; lower bearing surface of strut charred to depth of 1/8 in. and fibers crushed and bent over to north maximum of 1 in. on north side. Sides of column charred to depth of 1 1/8 in.; minimum section of unburned wood 9 3/4 by 9 1/2 in.; top of bearing surface of column very little charred, but discolored to depth of 1 in.; surface convex, rounded over from 1/2 to 1 in. at edges; fibers crushed and bent over mostly on east and west sides; wood quite soft to a depth of 1/4 in. Length before test, 11 ft. 2 in.; average after test, 11 ft. 1 1/4 in.; decrease in length, 3/4 in. (Figs. 82, 141, and 47).

XI. RESULTS OF FIRE AND WATER TESTS

1. APPLIED LOADS, DURATION AND EFFECT OF FIRE AND WATER

In the fire and water test, the column was subjected to working load, and to fire for a predetermined period not exceeding one hour, after which a hose stream was applied to three sides. On cooling, the column was either loaded to failure or subjected to an excess load equal to about twice the load sustained during the fire and water periods.

A general summary of applied loads, duration and relative intensity of fire exposure, duration and pressure of hose-stream application, and the general effects of fire and water are given in Table 44. Further details of columns and protections are given in Tables 4*a* and 4*b* (p. 51–54).

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2. PHOTOGRAPHIC RECORDS

Views of the columns in the fire and water series at several stages of the test are given in Figs. 83 to 89, Appendix A (p. 242–248).

3. FURNACE AND COLUMN TEMPERATURES

The temperatures observed in the furnace and test columns are given by the curves in Figs. 142 to 145, Appendix B (p. 303–306). The arrows on the plots indicate the time water was applied in each test.

4. LONGITUDINAL DEFORMATION

The expansion of the column during the fire period and contraction on application of water were determined by measurement of the movement of the head of the column, and the amounts are given in the respective test logs. These effects were quite small except in the case of the unprotected cast-iron columns, both of which attained maximum expansion before water was applied (Fig. 46, p. 124).

5. SUBSEQUENT LOADING TESTS

The loading to which the columns were subjected subsequent to the fire and water test are given in Table 44. They are loaded to their maximum sustaining capacity with the exception of four columns that were loaded to about twice their design working load and reserved for use in further tests, and the one protected by plaster on metal lath on which a subsequent fire test to failure under working load was made.

6. LOG OF FIRE AND WATER TESTS

(a) CONCRETE PROTECTIONS

TEST NO. 101. ROLLED H. 2-IN. CONCRETE PROTECTION. UPPER SECTION, LIMESTONE; MIDDLE SECTION, TRAP; LOWER SECTION, JOILET GRAVEL. TIED.

Fire Test.—30 mins.—no effect noted; flame very smoky. 32 mins.—vertical cracks on east and west faces near corners in lower 3 ft., ¼ in. wide at bottom. 38 mins.—⅛-in. vertical crack at center of south face, 2 ft. up. 45 mins.—cracks open $\frac{1}{16}$ in. to ⅛ in.; small spall on southwest corner, 3 ft. up. 60 mins.—gas shut off; no change in column except cracks had opened up slightly.

Water Test.—61¼ mins.—water applied to column through a 1⅛-in. nozzle; pressure at base of play pipe, 50 lbs. per sq. in.; nozzle was maintained at a distance of 20 ft. away from and to the west of column in all tests and the stream applied directly to the west face and at an angle to the north and south of faces; stream was played slowly up and down over all three faces of column. 61½ mins.—pieces washed off southwest corner and west face, 3 ft. up. 61¾ mins.—some pieces fell off west face in upper section. 62¼ mins.—pieces falling on west face and corners in middle section. 63¾ mins.—concrete fell in lower 2 ft., exposing steel. 64½ mins.—north

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AD TEST

	Ultimate Load, Lb.
	Not applied
	442000
	a537000
	Not applied
	b445000
	Not applied
	b348000
	b311000
	Not applied
lied	c Not applied
	379000
	423000
	536000
	527000
	502000

-in. nozzle. Stream

machine. The con-
middle and load re-

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face washed off to depth of 1½ in., 1 to 2½ ft. up; edge of flange exposed on northwest corner. 65¾ mins.—pieces fell off north face near top. 66¼ mins.—water shut off; duration of water test, 5 mins.

After Test.—Edge of flange exposed on northwest corner 1 to 2½ ft. up, and southwest corner for 4 in. at 5 ft. and at 8 ft. up; these corners spalled not over 1½ by 1½ in. for rest of length. Concrete in center of west face generally pitted from ⅛ to ¼ in. for the whole length; at bottom of upper section concrete pitted to depth of 1 in. for about 6 in. On south face all concrete washed off in lower 3 ft., exposing flange; about two-thirds of rest of surface pitted slightly. On north face surface generally smooth except at northwest corner. Fine vertical cracks in east face nearly full length, 1½ in. from corners. Column expanded 0.26 in. during the fire test and contracted 0.06 in. during the water test.

Load Test.—On the following day the column was subjected to a load of 234 000 lbs. this being calculated as the design dead load plus 2½ times the live load, assuming the former to be one-third of the latter. A faint cracking sound was heard at 69 000 lbs. At about 175 000 lbs. a dull thud was heard and several small pieces of concrete fell off northwest corner about halfway up. Column withstood test satisfactorily. The column recovered from the depression due to the loading within 0.002 in. on release of load. (Figs. 83 and 142.)

TEST NO. 102. ROLLED H. 2-IN. CONCRETE PROTECTION. UPPER SECTION, TRAP; MIDDLE SECTION, JOILET GRAVEL; LOWER SECTION, LIMESTONE. NO TIE.

Fire Test.—21 mins.—slight flaking at southeast corner, 9 ft. up. 41 mins.—column glowing all over. 60 mins.—gas shut off. No cracking or spalling.

Water Test.—62¼ mins.—water applied to column, 50-lb. pressure. 62¾ mins.—cracks noted on west face near both corners; steam very heavy, making observation difficult. 64 mins.—concrete fell on north in lower 10 ft., exposing steel. 65 mins.—long vertical crack at southwest corner, exposing flange in places in lower 10 ft. 67¼ mins.—water shut off; duration of water test, 5 mins.

After Test.—North flange exposed completely 1½ to 11 ft. up. South flange exposed on southwest corner for 2½ by 14 in. at 2½ ft. up, for 4 by 20 in. at 5 ft. up, for 3 by 10 in. at 8 ft., for 1½ by 12 in. at 10 ft. up; concrete loose on south flange from 1 to 11½ ft. up. On west concrete washed off to depth of ½ to 1½ in. at 3 ft., 4½ ft., 8 ft., and 10½ ft. above base; concrete least washed off on middle section; column expanded 0.26 in. during the fire test and contracted 0.09 in. during the water test.

Load Test.—On the following day the column in the unstripped condition was loaded until failure occurred at 442 000 lbs. by buckling to west, about 8 ft. above base. Scaling on the surface of the steel was first noted on the north flange at 373 000 lbs., the yield point of the steel as determined from the depression curve of the top of the column, being attained at 400 000 lbs. (Figs. 83 and 142.)

TEST NO. 103. PLATE AND ANGLE. 4-IN. CONCRETE PROTECTOIN. UPPER SECTION, TRAP; MIDDLE SECTION, GRANITE; LOWER SECTION, LIMESTONE. TIED.

Fire Test.—60 mins.—gas shut off; 1/16-in. vertical crack, 4 in. long, on west near southwest corner at bracket; no other cracking, spalling or other visible effects on column.

Water Test.—62 mins.—water applied to column; 50-lb. pressure. 62¾ mins.—small piece falling on west; face and corners rounded. 63½ mins.—continued falling of small pieces; steam heavy. 64½ min.—water effects most marked at bottom of middle section. 65½ mins.—continued falling of small pieces. 67 mins.—water shut off; duration of water test, 5 mins.

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After Test.—Little effect on north and south faces back of corners except in middle section, where granite concrete was pitted to depth of ¼ in. quite generally. On west face the middle section was most affected and the upper section least affected; concrete in middle section quite generally washed away to depth of 1½ in. at center, and 3½ in. at corners; in upper and lower sections, ¼ to ½ in. at centers and 3½ in. at corners; concrete in middle section cracked along flanges near each corner; steel flanges not exposed. East face unaffected. Column expanded 0.15 in. during the fire test.

Load Test.—On the following day the column in the unstripped condition was loaded to 547 000 lbs., the capacity of the machine, without signs of distress or failure. Column was then stripped from 5 ft. 2 in. to 7 ft. 10 in. above base and loaded until failure occurred at 537 000 lbs. by buckling to west, 7 ft. 8 in. above base. Scaling of steel was first noted at 433 000 lbs. and decided yielding began at 475 000 lbs. (Figs. 83 and 142.)

TEST NO. 104. PLATE AND ANGLE. 2-IN. CONCRETE PROTECTION. UPPER SECTION, CINDER; MIDDLE SECTION, SANDSTONE; LOWER SECTION, TRAP. TIED.

Fire Test.—31 mins.—fine vertical crack, 24 in. long, on west near southwest corner, 6½ ft. up; also on east, southeast corner, 4½ ft. to 7 ft. up; slight crushing on southeast corner 8 ft. up, small piece falling at 33 mins. 35 mins.—¼-in. crack on center of south face, 6½ to 9½ ft. up, extending to southwest corner at upper end. 50 mins.—cracks previously noted opening up ¼ to ½ in. and extending in length in middle section. 53 mins.—piece spalled off northeast corner, 7 ft. up. 60 mins.—gas shut off; only cracks are the three noted in middle section, 3 to 4 ft. long, and ¼ to ½ in. wide; no cracks on north.

Water Test.—61½ mins.—water applied to column; 50-lb. pressure. 61¾ mins.—concrete fell off on south 5 to 8 ft. up, exposing steel. 63 mins.—concrete in lower 4 ft. on west washed and pitted considerably. 63½ mins.—concrete on northwest corner fell, 4 to 7 ft. up. 64¾ mins.—upper 4 ft. on west pitted. 65 mins.—piece fell on northwest corner, 10 ft. up. 66 mins.—pieces washed off corners on west in lower 4 ft. 66½ mins.—water shut off; duration of water test, 5 mins.

After Test.—West face pitted ½ to 2 in. in lower section, ¾ in. in middle section, and ½ to 2 in. in upper section; northwest corner off for full length, exposing flange edge for 1¼ by 12 in. at 2 ft. up, for 2 by 12 in. 4 ft. up, and for 1½ in. 5 to 9 ft. up; north face intact except for spalling on northwest corner; southwest corner spalled full length exposing flange edge for 3 by 4 in. 2 ft. up, and 2 by 6 in. 6 ft. up; entire south flange exposed in middle section; east side intact except for a few fine cracks. Column expanded 0.26 in. during the fire test and contracted 0.09 in. during the water test.

Load Test.—On the following day the column in the unstripped condition was subjected to an excess load of 234 000 lbs. without developing any signs of distress and with full recovery from the deformation on removal of load. (Figs. 84 and 142.)

(*b*) HOLLOW CLAY-TILE PROTECTIONS

TEST NO. 105. PLATE AND ANGLE. 2-IN. HOLLOW CLAY-TILE PROTECTION, UNFILLED. UPPER SECTION, SURFACE CLAY, BOSTON DISTRICT; MIDDLE SECTION, SEMIFIRE CLAY, NEW JERSEY DISTRICT; LOWER SECTION, OHIO SHALE. NO FILLING. OUTSIDE WIRE TIES.

Fire Test.—2 to 3 mins.—vertical cracks and spalling of parts of outer shells on north and west sides in lower section. 3 to 20 mins.—continued cracking, spalling, and bulging of outer shells in lower section; tile in upper half very little affected. 30 mins.—

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a little more spalling in lower section; vertical cracks on west at bracket, and at south and west 5 ft. up, ⅛ in. wide. 39 mins.—outer shells spalled off on east and west, 3d course, shell hanging loose on east, 2d course. 45 mins.—gas shut off; very little damage to middle and upper sections except for vertical cracks as noted; lower three courses cracked and spalled.

Water Test.—46 mins.—water applied to column; 30-lb. pressure. 46½ mins.—all tile washed off on west 1 to 2 ft. up, and on north 1 to 3 ft. up, exposing steel. 47 mins.—all tile down in lower sections on north and west 1 to 4 ft. up. 47½ mins.—tile down on north and west 4 to 5 ft. up. 48 mins.—tile still in place on south except outer shell spalled 3 to 4 ft. up; tile down on west in center 5 to 6 ft. up. 48½ mins.—water shut off; duration of water test, 2½ mins.

After test.—Tile down and steel exposed on north and east 1 to 5 ft. up and on west 1 to 6 ft. up; tile loose on south 1 to 5 ft. up. Tile in upper half of column almost intact except outer shell cracked on west 6 to 7 ft. up, and part of outer shell off on north at bracket. Ties in lower five courses fell down, other ties in place. Expansion during fire test, 0.22 in.; contraction during water test, 0.13 in.

On the following day the column was stripped. Mortar in joints found to be fairly full, although in about one-half of the joints it had been washed out to depth of ½ to ⅝ in. Mortar on flanges broke away from the steel and adhered to the tile. Lateral deflection less than ⅛ in.

Load Test.—The stripped column was loaded until failure occurred at 445 000 lbs. with buckling to east about 6 ft. above base. (Figs. 84 and 143.)

TEST NO. 106. PLATE AND ANGLE. 2-IN. HOLLOW CLAY-TILE PROTECTION. UPPER AND LOWER SECTIONS, OHIO SEMI-FIRE CLAY; MIDDLE SECTION, SURFACE CLAY, CHICAGO DISTRICT. OUTSIDE WIRE TIES IN UPPER HALF OF COLUMN, WIRE MESH IN JOINTS IN LOWER HALF. CONCRETE FILLING.

Fire Test.—2½ to 6 mins.—outer shells spalling at corners in middle section. 9 to 12 mins.—outer shells shattered on west in middle section, but did not spall off; ⅛-in. cracks near corners on west in upper section. 12½ to 18½ mins.—outer shells spalled and bulging in middle section; at end of this period, all of outer shells were down in 5th course on north and west and partly down on east. 20½ mins.—vertical crack in upper section on west 9½ to 11½ ft. up, ⅝ in. wide, opening to 1 in. at 26 mins. 23 mins.—outer shell buckled on east, 9 ft. up. 25½ mins.—tie wire broken on 10th course. 31 to 36 mins.—continued cracking and spalling of outer shells in middle section, also bulging out on north 7 ft. up. 38 mins.—outer shell spalled on 6th course on north. 40 mins.—all outer shells spalled on east 3 to 5 ft. up. 44 mins.—outer shells on east 7 ft. up buckle out 2 in. 45 mins.—gas shut off; outer shells are off on 5th, 8th, and 9th courses on west, and on 5th and 6th courses on north; column expanded 0.24 in. during the fire test.

Water Test.—47½ mins.—water applied to column; 30-lb. pressure. 47¾ mins.—considerable amount of tile fallen; steam obscures view. 48½ mins.—tile down in middle 8 ft. on north, south and west, exposing flanges on north and south; concrete filling in place. 49¾ mins.—water off; duration of water test, 2¼ mins.

After Test.—Steel exposed on north 1 to 11 ft. up, on south from 1 to 10 ft. up except where parts of inner shells remained in place; wire mesh still in place on north at 1 ft., 4 ft., 5 ft. and 6 ft. up. On west tile off, exposing concrete 2 to 10 ft. up; outer shell off 10 to 12 ft. up. On east side, concrete exposed 2 to 4 ft. up; the rest was covered by tile or inner shells of same. Tile all in place on bottom course on all sides and on bracket course on north, east, and south sides.

Load Test.—On the following day the column was subjected to an excess load of 228 000 lbs. without developing any signs of distress and recovered from deformation almost completely on removal of load. (Figs. 85 and 143.)

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TEST NO. 107. PLATE AND CHANNEL. 4-IN. HOLLOW CLAY-TILE PROTECTION. UPPER SECTION, OHIO SHALE; MIDDLE SECTION, SEMIFIRE CLAY, NEW JERSEY DISTRICT; LOWER SECTION, SURFACE CLAY, BOSTON DISTRICT. OUTSIDE WIRE TIES ON UPPER HALF OF COLUMN; WIRE MESH IN JOINTS IN LOWER HALF. NO FILLING.

Fire Test.—12 mins.—no cracking or spalling noted. 15 mins.—vertical cracks on east from 8 ft. up to top, ¼ in. wide; part of outer shell spalled on 11th course, east; $\frac{1}{16}$-in. vertical crack on east 6 to 8 ft. up. 19 mins.—$\frac{1}{16}$-in. vertical crack on west 6 to 9 ft. up. 22 mins.—cracks on east and west near top, ½ to 1 in. wide. 25 to 27 mins.—outer shells on west bulging out 1½ to 2 in. in upper section, held by ties. 31 to 35 mins.—¼-in. vertical crack on west at south corner from 9 ft. up to top; other cracks opening up and outer shells bulging in upper section. 42 mins.—continued cracking in upper section; new crack on north 1 in. wide, 11th course; tile on east 11th and 12th courses cracked. 45 mins.—gas shut off; tile in upper section generally cracked and outer shells loose; lower section practically intact; middle courses have a few vertical cracks as noted.

Water Test.—46 mins.—water applied to column; 30-lb. pressure. 46½ mins.—little effect; outer shell on west fractured in upper section. 46¾ mins.—little apparent change. 47¾ mins.—outer shell down on west 9 to 11 ft. up. 48¼ mins.—tile almost all down on upper three courses. 48½ mins.—water shut off; duration of water test, 2½ mins.

After Test.—All tile down in 10th, 11th, and 12th courses, except on north side, where tile is held by bracket; some mortar adheres to north and south flanges. Tile shattered on east and west, in 9th course. Tile in good condition in lower 8 ft. except for a few ⅛-in. vertical cracks on east and west 6 to 8 ft. up, and small hole in outer shell on west 7 ft. up. Mortar and mesh in joints in lower courses in good condition, it being necessary to wedge the tile apart in order to remove the covering. Column expanded 0.15 in. during the fire test and contracted 0.10 in. during the water test. Maximum deflection after fire and water test, $\frac{1}{32}$ in. to west.

Load Test.—On the following day the column was stripped and loaded to failure at 348 000 lbs., column buckling to west about 6 ft. above base. Decided yielding of the steel began at 322 000 lbs. (Figs. 85 and 143.)

(*c*) GYPSUM-BLOCK PROTECTIONS

TEST NO. 108. ROLLED H. 2-IN. SOLID GYPSUM-BLOCK PROTECTION. UPPER HALF, WESTERN GYPSUM; LOWER HALF, EASTERN GYPSUM. FILLED.

Fire Test.—13 mins.—fine surface checks beginning to develop. 28 mins.—surface checks generally ⅛ in. wide and ⅜ in. deep near lower end of column to very fine at top. 36 mins.—joints at bottom open $\frac{1}{16}$ to ⅛ in. 43 mins.—very little change in fire effects. 45 mins.—gas shut off.

Water Test.—46 mins.—water applied to column; 30-lb. pressure. 47¾ mins.—little effect noted except for surface erosion on west. 48 mins.—blocks fell on west 2 ft. 10 in. to 5 ft. 8 in. up. 48¼ mins.—blocks down on west 1½ to 9 ft. up, exposing flange edges and filling. 48½ mins.—blocks down on south, 4¼ to 7 ft. up. 48¾ mins.—blocks down, exposing steel nearly full length on north and south. 49 mins.—water off; duration of water test, 3 mins.

After Test.—All blocks down on north 1½ to 9 ft. up, exposing steel; on south, from 1½ ft. up to top of bracket, exposing steel except for a 2 ft. length above middle of column, where mortar remained in place; on west, 1½ to 4¼ ft., 6 to 9 ft., and from 11 ft. to top. Filling washed out on west, exposing web from 2 ft. 4 in. to 3 ft. up. All blocks in place on east except at places on corners. Blocks remaining on column on west washed off to a depth of ¾ in. to point where fiber in blocks was not charred.

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Mortar on flanges, and filling in web had evidently been quite full. Steel not rusted except where exposed by water. Column expanded 0.05 in. during the fire test and contracted 0.05 in. during the water test. Maximum lateral deflection after fire and water tests, $\frac{1}{16}$ in.

Load Test.—On the following day the column was loaded to failure at 311 000 lbs. column buckling to west about 7 ft. above base. (Figs. 86 and 143.)

TEST NO. 109. ROLLED H. 4-IN. SOLID GYPSUM-BLOCK PROTECTION. UPPER SECTION, EASTERN GYPSUM; LOWER SECTION, WESTERN GYPSUM.

Fire Test.—16 mins.—a few corner cracks developed in lower half, $\frac{1}{16}$ by 1 in. 22 mins.—fine surface checks beginning to develop near bottom. 42 mins.—surface checks quite general, forming in $\frac{3}{8}$-in. squares, checks $\frac{1}{32}$ in. wide at bottom and $\frac{1}{64}$ in. wide at top; corner cracks opening to maximum of $\frac{1}{8}$ in.; vertical joints in lower half open $\frac{1}{32}$ in.; slight spalling at corners. 53 to 57 mins.—surface checks open from $\frac{1}{8}$ in. at base to $\frac{1}{32}$ in. at top of column. 59 $\frac{1}{2}$ mins.—gas shut off; surface checked all over, most at bottom; otherwise covering and column little affected.

Water Test.—60$\frac{3}{4}$ mins.—water applied to column; 50-lb. pressure. 61$\frac{3}{4}$ mins.—hose burst, water off; gypsum washed off on west to depth of about 1 in., mortar in joints washed out from 1 to 4 in. depth, exposing ends of ties. Gypsum washed off on north and south sides, $\frac{3}{4}$ to 1 in. depth; east side intact. 72$\frac{1}{2}$ mins.—water again applied; 50-lb. pressure. 73$\frac{1}{2}$ mins.—outer 1 in. of gypsum washed off clean on north and south. 74 mins.—almost all of mortar washed out of vertical joints on west. 76$\frac{1}{4}$ mins.—$\frac{1}{8}$-in. vertical crack on west near south corner 9 to 10 ft. up. 76$\frac{1}{2}$ mins.—water shut off; total duration of water test, 5 mins.

After Test.—On west side surface washed very smooth, corners rounded; blocks slightly less than 3 in. thick; no mortar in vertical joints; ties all in place and hold mortar in horizontal joints; on north and south sides, faces of blocks rough, with fibers projecting; blocks about 3 in. thick; all joints generally full; east side little affected; no blocks loose; edges of bracket and stiffner angles exposed. Column expanded 0.04 in. during the fire test; less than 0.01 in. contraction during the water test.

Load Test.—On the following day the column in the unstripped condition was subjected to an excess load of 234 000 lbs., which it withstood without any signs of distress except for a few small vertical cracks in the blocks. (Figs. 86 and 143.)

(*d*) PLASTER ON METAL-LATH PROTECTION

TEST NO. 110. PLATE AND ANGLE. TWO LAYERS OF PORTLAND-CEMENT PLASTER ON METAL LATH WITH $\frac{3}{4}$-IN. AIR SPACE BETWEEN LAYERS.

Fire Test.—6 mins.—furnace gases heavy, difficult to see column. 18 mins.—fire more luminous. 19 mins.—plaster cracked and spalled on all sides at bottom of bracket; this probably occurred at about 14 mins.; corners spalled slightly on west 32 mins.—fine crack on east, 2 in. long, 4 ft. up. 45 mins.—gas shut off; several fine cracks noted on all faces.

Water Test.—46$\frac{1}{4}$ mins.—water applied to column; 30-lb. pressure. 46$\frac{3}{4}$ mins.—vertical cracks near corners on all faces about 6 ft. long. 47$\frac{1}{4}$ mins.—piece of plaster, 8 in. square, spalled off north face, 6 ft. up. 48 mins.—plaster washed off southwest corner in upper and lower 3 ft. 48$\frac{3}{4}$mins.—plaster washed off northwest corner, lower 4 ft. 49$\frac{1}{2}$ mins.—water off; duration of water test, 3$\frac{1}{4}$ mins.

After Test.—On west side plaster off, 1 by 1 in., exposing lath on northwest corner 1 to 4 ft. up. Fine horizontal cracks across face at 4 ft., 6 ft., 7 ft., 7$\frac{1}{2}$ ft., 8 ft., 9$\frac{1}{2}$ ft., and 11 ft. up. On north side, lath exposed near west corner, 5 by 18 in. 6 ft. up. On south side, lath exposed on southwest corner, 1 by 2 in., from 7 ft. up to top; fine

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horizontal cracks across face at 5 and 8 ft. up. On east side, vertical crack runs full length near south corner; fine horizontal cracks at 2½, 4, and 6 ft. up. Plaster cracked and lath exposed on all sides at bottom of bracket. Column expanded 0.14 in. during the fire test. No contraction during the water test (see Fig. 46).

Subsequent Fire Test.—Since the protection was little injured by the fire and water test, it was subjected to a second fire test to failure on the day following the fire and water test. 5 mins.—plaster bulging out 2 in. on north at bottom of bracket. 7 mins.—crack on west at southwest corner open ½ in., 7 to 8 ft. up, extending further 4 ft. vertically at 11 mins. 14 mins.—bulging of plaster on southeast below bracket. 39 mins.—plaster bulging away from lath in places on southwest corner from 7 ft. up to top. 46 mins.—northeast corner cracks opening up, 8½ to 11 ft. up. 1 hr.—bulging of plaster increasing on all faces at bracket. 1 hr. 54 mins.—no change except all cracks opening up slightly. 2 hrs. 25 mins.—about one half of lath exposed at bracket on west. 2 hrs. 28 mins.—small areas of lath exposed at bracket on east. 2 hrs. 47¼ mins.—failure with buckling to west, maximum at 7½ ft. above base. Column expanded maximum of 0.92 in. at 2 hrs. 30 mins.; from this point to one minute before failure it compressed ½ in. (Figs. 87, 144, and 46.)

(e) REINFORCED-CONCRETE COLUMNS

TEST NO. 111. SQUARE VERTICALLY REINFORCED CONCRETE. UPPER SECTION, LIMESTONE; MIDDLE SECTION, MERAMEC RIVER GRAVEL; LOWER SECTION, LIMESTONE.

Fire Test.—14 to 24 mins.—slight spalling of small pieces from corners in middle section. 26 to 31 mins.—cracking of concrete in middle section on all sides, cracks $\frac{1}{16}$ to $\frac{1}{8}$ in. wide, and 12 to 30 in. long; cracking and crushing at southwest corner 6½ ft. up, cracks extending diagonally downward on both south and west faces. 36 mins.—upper and lower sections apparently unaffected. 36½ mins.—crack on south 4½ ft. up, extending downward to 2 ft. above base. 37 mins.—cracks at southwest 6½ ft. up, now open ⅜ in. 46 min.—same crack open 1½ in., new crack extending upward 12 in. from same; crack on east, 5½ ft. up open ⅜ in. 48 mins.—small vertical and diagonal cracks extending from crack at southeast corner 4½ to 5½ ft. up. 53 mins.—$\frac{1}{16}$ in. vertical and diagonal cracks near northwest corner on north and west 5 ft. up. 60 mins.—gas shut off; very little spalling; concrete at southwest corner in middle section, crushed.

Water Test.—61½ mins.—water applied to column; 50-lb. pressure. 63 mins.—steam heavy; observations impossible. 63¼ mins.—concrete spalled to reinforcing rod on northwest corner in lower half. 63½ mins.—west face washed off to about 1½-in. depth. 64 mins.—northwest rod exposed to 10 ft. up. 64½ mins.—southwest rod exposed to 11 ft. up. 65 mins.—concrete washed off in middle of column on west, exposing tie. 65¾ mins.—little change during past ¾ min. 66½ mins.—water off; duration of water test, 5 mins.

After Test.—Concrete washed off both corners of west face 4 by 4 in., exposing reinforcing rods 1 to 11 ft. up; in center of west face concrete is washed off to average depth of 1 in. in upper section, 2 in. in middle section, exposing ties, and ⅛ in. in lower section; on north, east, and south faces, concrete in place in upper and lower sections except at west corners; in middle section, concrete washed off south face to depth of 1 in., 4½ to 5½ ft. up, and off north face to depth of ½ in. in places from 4 to 6 ft. up; east side unaffected by water, but has several vertical cracks in middle section from the fire test. Estimated minimum area of cross section of concrete not spalled or loose in middle section, 187 sq. in., a decrease of a little more than 25 per cent of the original section. Column expanded 0.28 in. during the fire test and contracted 0.09 in. during the water test.

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Load Test.—On the following day the column was loaded to failure at 379 000 lbs.; column failed by diagonal shearing from a point 4½ ft. up on southeast corner to a point 8½ ft. up on northwest corner; no cracking or yielding was noted before the maximum load was applied. (Figs. 88 and 145.)

TEST NO. 112. ROUND VERTICALLY REINFORCED CONCRETE. UPPER SECTION, LIMESTONE; MIDDLE SECTION, MERAMEC RIVER GRAVEL; LOWER SECTION, JOLIET GRAVEL.

Fire Test.—18 mins.—flaking on southeast corner, 6 ft. up. 23 to 25 mins.—flaking in middle section on south and east. 30 mins.— concrete crushed on south, 6 to 7½ ft. up, cracks opening to ½ in. at 32 mins.; loose pieces ready to fall. 31 mins.—vertical crack on southwest ¼ in. wide, 4 to 7½ ft. up. 39 mins.—crack on south, 6 to 7 ft. up, open ⅜ in. 43 mins.—several new cracks ⅛ in. wide and about 12 in. long in middle section. 51 to 57 mins.—spalling in middle section to depth of 1½ in.; upper and lower sections intact. 60 mins.—gas shut off.

Water Test.—61¼ mins.—water applied to column; 50-lb. pressure. 62¼ mins.—concrete washed off, exposing tie on west, middle section. 62½ mins.—some concrete fell on south, middle section. 63½ mins.—concrete down on south, 2 to 5 ft. up. 64 mins.—reinforcing bars exposed on west in lower 7 ft. 64½ mins.—piece fell on west, 7 ft. up. 66 mins.—piece fell on south, 8 ft. up; not much change during past 2 mins. 66¼ mins.—water off; duration of water test, 5 mins.

After Test.—Concrete off to reinforcing bars from 3 to 8½ ft. up on west and from 3 to 8 ft. up on south; also piece off, 1 by 6 by 12 in., at 12 ft. up, and 1 by 4 by 5 in., at 11 ft. up on west. On north concrete loose to outside of ties, 2 to 6 ft. up and piece off, 1½ by 12 by 6 in., at 6½ ft. up; on east, crack ½ in. wide from 6 to 9½ ft. up; concrete shattered near middle; crack on northeast corner, ¼ in. wide, 1½ to 4½ ft. up; several smaller horizontal and vertical cracks; section at middle of column reduced about 30 per cent due to spalled or loose concrete. Column expanded 0.14 in. during the fire test and contracted 0.02 in. during the water test.

Load Test.—On the following day, the column was loaded to failure at 423 000 lbs.; failure occured by diagonal shearing from 5 ft. up on north to 7 ft. up on south; cracking sounds were heard at about 290 000 lbs., which increased as failure was approached. (Figs. 88 and 145.)

TEST NO. 113. HOOPED REINFORCED CONCRETE. UPPER SECTION, TRAP; MIDDLE SECTION, MERAMEC RIVER GRAVEL; LOWER SECTION, GRANITE.

Fire Test.—9 to 20 mins.—flaking and cracking in middle section, cracks ¼ in. maximum, 2 to 3 ft. long. 21 mins.—piece ½ by 4 by 4 in. spalled on southwest, 5 ft. up. 22 mins.—crack on southwest in middle section open ¾ in. 23 mins.—flaking and buckling of concrete shell general in middle section. 27 mins.—piece 1 by 4 by 10 in. spalled on southwest, 4½ ft. up. 28 mins.—piece spalled, 1¾ by 10 in. on south 4 to 7 ft. up. 34 mins.—buckling of concrete shell in middle section to 2 in. maximum. 37 mins.—upper and lower sections intact except for cracks extending from middle section. 38 mins.—spalling on south, 4 to 8 ft. up; spiral exposed for 8 turns on 10-in. width. 43 mins.—concrete shell on west middle section, standing out 2 in. 43½ mins.—spalling on east, 4 to 8 ft. up; spiral exposed for 16 turns on 5-in. width. 46 mins.—cracks on west extending down to 1 ft. from base, are ½ in. wide at 3 ft. up, opening to 1½ in. at 52 mins. 58 mins.—cracks on east in middle section, extending down to 3 ft. above base; finer cracks below. 60 mins.—gas shut off.

Water Test.—61¼ mins.—water on, 50-lb. pressure. 61½ mins.—hose burst; water shut off; concrete stripped to spiral on west from 1 to 8½ ft. up and partly on north and south 2 to 6 ft. up; above this on north and south concrete off to within 1 in. from spiral; aggregate split on plane of spiral. 79½ mins.—water again applied. 80¼

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mins.—pieces fell on west, exposing spiral at 6½ ft. up. 80¾ mins.—pieces fell on west, exposing spiral in upper section. 82 mins.—one-half of the circumference of the spiral exposed on west up to 9 ft. above base and one-fourth of its circumference exposed above this point; little change during past two minutes except concrete washed out between wires. 84¼ mins.—water shut off; total duration of water test, 5 mins.

After Test.—Concrete washed off to spiral all around in middle section, 4 to 8 ft. up. In lower section spiral exposed all around 1 to 4 ft. up, except for strip about 12 in. wide on east. In upper section, spiral exposed for about half of circumference on west, 8 to 10½ ft. up and for about one-fourth of circumference up to 12 ft. above base. Column expanded 0.14 in. during the fire test and contracted 0.045 in. during the water test.

Load Test.—On the following day the column was loaded until failure occurred at 536 000 lbs., 6½ ft. above base; all bars buckled out 1¼ in., and the spiral wire broke in tension with reduced section; faint cracking sounds were first heard at about 300 000 lbs.; at 400 000 lbs. the compressive deformation due to a given load increment was about twice that at the start, and at 500 000 lbs. about five times the initial rate. (Figs. 89 and 145.)

(*f*) UNPROTECTED CAST-IRON COLUMNS

TEST NO. 114. ROUND CAST IRON. VERTICALLY CAST; UNPROTECTED

Fire Test.—22½ mins.—gas shut off; column glowing low red in lower half; no visible deflection.

Water Test.—23 mins.—water applied to column; 30-lb. pressure. 23¼ mins.—column still glowing on north, east, and south. 23¾ mins.—still glowing on east. 24 mins.—water off; duration of water test 1 min.; column still glowing in lower third on east.

After Test.—Column unaffected as far as could be seen except for deflection of 1⅜ in. to west, 4 ft. above base. Column expanded 0.99 in. during the fire test and contracted 0.51 in. during the water test. (See Fig. 46.)

Load Test.—On the following day the column was loaded to failure and supported a maximum load of 527 000 lbs.; pumping was continued, but load fell off gradually until at 428 000 lbs. column broke in two with great violence 6 ft. above base; during the loading the column deflected ⅛ in. to west at center at 445 000 lbs., and ½ in. at 520 000 lbs. fracture coarse, gray, crystalline; break very jagged. (Figs. 89, 146, and 46.)

TEST NO. 115. ROUND CAST IRON. VERTICALLY CAST; UNPROTECTED

Fire Test.—20 mins.—trace of color visible on column. 30 mins.—gas shut off; column glowing low red entire length; no visible deflection.

Water Test.—31¼ mins.—water applied to column; 30-lb. pressure. 31⅝ mins.—column still glowing on north, east, and south. 31¾ mins.—still glowing on east. 32 mins.—still glowing in lower half on east. 32¼ mins.—water off; duration of water test, 1 min.; no color on column.

After Test.—Column unaffected as far as could be seen except for deflection of ⅞ in. to west, 5½ ft. above base; column expanded ⅞ in. during the fire test and contracted ⅝ in. during the water test. (See Fig. 46.)

Load Test.—On the following day the column was again loaded and supported a maximum load of 502 000 lbs.; pumping was continued with load decreasing, until at 348 000 lbs. it was removed to avoid breaking the column; during the test, column deflected ¾ in. to east at center at 290 000 lbs., when deflection changed direction; at 450 000 lbs. it was ⅛ in. east, increasing rapidly to the west, until at maximum load it was 1½ in. west; deflection continued to increase until load was removed, when column recovered a large part of the deflection. (Figs. 89, 145, and 46.)

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XII. GENERAL SUMMARY AND DISCUSSION

Included under this head are considerations relative to the quality of the materials of the test columns and their coverings; the general effects of load, fire, and water; the useful limit of the various types of columns as loaded and exposed to fire; test durations, effects of methods of application, and causes of variation in results.

1. CHARACTERISTICS OF COLUMNS AND THEIR MATERIALS

These relate to the structural quality of the test columns and the physical and thermal effects of load and fire.

(a) STRUCTURAL-STEEL COLUMNS

(1) MATERIAL AND FABRICATION.—The structural steel for the test columns was made by the open-hearth process in four mills in different localities, and in point of chemical and mechanical properties came fairly within accepted specification limits. Measurements of area of structural sections indicated general agreement with the nominal or handbook areas within 1 per cent, although a few measured areas differed from the nominal by amounts up to 4 per cent.

The columns were detailed according to standards of current practice, and as furnished fabricated by five companies no defects due to faulty design or fabrication were noted before test or developed as a result of the tests, except that the bearings as received were too uneven to insure uniform distribution of load.

A number of structural-steel columns that had been subjected to fire and water tests in the protected condition without injury to the steel were subsequently stripped of their covering and loaded to failure, the maximum sustained being 30 600 to 37 600 pounds per square inch, which represent ultimate factors of safety from a little less than 3 to over 4 on the computed working loads.

(2) EFFECT OF SLENDERNESS RATIO.—The slenderness ratio $\left(\frac{l}{r}\right)$ within the limits 40 to 80 appears to have little influence on the strength of structural-steel building columns as exposed to fire. In the case of some of the columns of higher slenderness ratio deflections were larger at a given stage of the test than for columns of more rigid section. The columns had flat end bear-

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ings, the upper end being fully restrained and the lower end partly restrained by the base angles and anchor bolts.

(3) LATERAL DEFLECTION.—The direction of the lateral deflection before and after failure conformed with the line of least rigidity of the section, except in a few cases where the direction of the deflection was influenced by local heating of portions of the column. Deflections immediately before failure of a little over 2 inches were noted in a few tests, although generally they were between 1½ and 2 inches. Decided lateral deflection did not, as a rule, develop until the point of maximum expansion had been passed.

(4) VERTICAL DEFORMATION.—Steel columns when loaded and exposed to fire expand up to a point where the rate of compression of the metal due to the load becomes equal to or greater than the thermal expansion. The total expansion varied from ⅞ of an inch for the columns nearly uniformly heated over the full exposed length to ⅛ of an inch for those subjected to local heating due to failure of portions of the covering.

The measured expansion per unit length up to the point of maximum expansion, as measured over a 37-inch gage length, varied from 0.0044 to 0.0066, the lower values being due mainly to local heating. The unit expansion per °C. rise of temperature, or coefficient of expansion of the columns as loaded and exposed to fire, averaged 0.0000125 (0.000007 per °F.) as taken from room temperature up to the point where decided yielding of the metal was apparent.

(5) LOAD CARRIED BY THE COVERING.—On application of working load, the covering takes portions of the load proportionate to its area and rigidity with reference to the steel. With increase of temperature the higher rate of expansion of the steel causes a larger portion of the load to be transferred to the steel section, this condition continuing up to the point of maximum expansion. Subsequently the compressive yielding of the steel causes load to be again transferred to the covering, the amount depending on the stability and load-carrying capacity retained by the covering materials after the fire exposure.

(6) AVERAGE EFFECTIVE TEMPERATURES.—For the steel columns for which average effective temperature determinations were made (Table 43), the range at maximum expansion was

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from 484 to 593° C (903 to 1099° F), with an average of 530° C (986° F). At failure the average temperature range was from 570 to 837° C (1058 to 1539° F), the average of all determinations being 668° C (1234° F). The applied working loads if carried by the steel alone correspond to stresses of 8900 to 14 500 pounds per square inch; as varying for the different structural sections, the average being 11 600 pounds per square inch (Table 41, p. 97).

The lower temperatures given above correspond nearly with those that hold for steel at maximum expansion and at failure under the given unit loads. The higher temperatures obtained in the tests where the covering carried portions of the load, this effect being much less marked at maximum expansion than at failure.

(7) GENERAL CAUSE OF FAILURE.—The failure in the fire test was due in all cases to decrease in mechanical strength of steel with increase of temperature. The temperature required to cause failure depended mainly on the unit load carried by the structural section, although uneven stress distribution as caused by incidental eccentricity of load application, uneven bearings, and deflection of the column, entered as possible modifying conditions. The general or local lateral deflections occurring immediately before failure were due to yielding of the metal and can be considered as failure effects.

The deflection and distortion at failure caused large permanent loss of load-carrying capacity, depending on the amount of the deflection and the rigidity of the section, the remaining strength being estimated at 5 to 50 per cent of that before test.

(b) CAST-IRON COLUMNS

(1) MATERIAL AND MANUFACTURE.—Of the 10 cast-iron columns tested, 7 were cast horizontally and 3 were cast in vertical position. The iron of the horizontally cast columns conformed with published specifications for gray-iron castings in point of chemical and physical properties, the specimens being cut from the ribs in the head section of the columns. The tests made on the iron of the vertically cast columns were too few to be conclusive, but they indicated difference in the strength of the iron of 15 to 20 per cent as between the two ends of the column (Table 9, p. 339).

The wall thickness of the columns differed from the nominal thickness by a maximum of ¼ of an inch for the horizontally

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cast columns and $\frac{1}{16}$ of an inch for the vertically cast pipe columns, the difference being caused mainly by displacements of the molding core in casting the columns.

Two of the vertically cast columns that had been subjected to fire and water tests, which induced permanent lateral deflections in one of $\frac{7}{8}$ inch and in the other of $1\frac{3}{8}$ inch, were subsequently loaded to failure, the maximum sustained being 36 400 and 34 700 pounds per square inch, respectively.

(2) DEFORMATION AND TEMPERATURE.—The same general characteristics of deformation and temperature obtained for the cast-iron as for the steel columns. The average unit expansion attained by the cast-iron columns was 0.0064 against 0.0054 for the steel columns, the average temperature at maximum expansion and at failure being about 70° C (126° F) higher than as obtained with the steel columns, the difference being due to the lower allowable working loads applied to the cast iron. The average applied unit loads, if assumed carried by the metal section alone was 6500 pounds per square inch as compared with 11 600 pounds per square inch average for the steel columns, a difference that was considerably reduced by the interaction of the coverings, which were heavier around the steel columns and generally carried larger proportions of the applied loads than those of the cast-iron columns.

(3) CAUSE AND CHARACTER OF FAILURE.—Failure in the fire test was primarily due to inability of the metal to sustain load at the given temperature, the bucking and fracture incident with failure being in the nature of failure effects. The partial or full fracture of the metal section at one or more points, resulted in almost complete loss of load-sustaining capacity. The direction of the deflection for the columns having uneven wall thickness was quite uniformly toward the side with the heavier thickness, due evidently to compressive yielding of the thinner and more highly stressed metal on the opposite side.

(c) PIPE COLUMNS

(1) MATERIAL AND MANUFACTURE.—No tests of the metal of the pipes were obtained, but the material was apparently mild steel. The metal was of standard thickness. Cylinders taken of the 1:1½:3 concrete mixed for the filling of the plain pipe

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column developed an average compressive strength of very nearly 4000 pounds per square inch at the time the column was tested.

The columns were filled at the manufacturer's plant and furnished complete with bearing details, the latter being arranged to approximate conditions of use in buildings (Fig. 7, p. 24).

(2) DEFORMATION AND TEMPERATURE.—The metal being exposed, attained early in the fire test a higher temperature than the filling, and expanding away from the latter, would assume most of the applied load, the unit stress, if all of the load was carried by the pipe metal, being 16 500 pounds per square inch for the plain concrete-filled pipe column. The point of maximum expansion was in this case well defined and was attained at an earlier period and lower temperature than in tests of steel and cast-iron columns of about the same duration, due to the higher load sustained by the metal. The compressive deformation subsequent to maximum expansion was large and developed the strength of the concrete filling before failure occurred.

The reinforced-pipe column which had structural angles embedded in the concrete filling, expanded only a small amount during the fire test, yielding of the pipe metal under the high induced stresses beginning early in the test. Toward the end of the test the temperature of the pipe was so high that it could have sustained only a small part of the load, which must have been carried mainly by the steel reinforcement with stresses too high to permit much expansion.

Lateral deflections began at about the time the pipe metal began to yield and increased as failure was approached.

(*d*) REINFORCED-CONCRETE COLUMNS

(1) MECHANICAL PROPERTIES OF THE CONCRETE.—Made under conditions approximating those obtaining in building construction, the concrete developed wide variability in strength and elastic properties, the principal cause of which was difference in water content of the concrete mixture. The average compressive strength of 8 by 16 inch cylinders of 1:2:4 limestone and of trap-rock concrete made from the concrete mixed for the reinforced-concrete columns was 1525 pounds per square inch at 28 days and 1930 pounds per square inch at 16 months, with maximum variations above and below the averages of 67 and 54 per

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cent, respectively. The modulus of elasticity at 650 pounds per square inch had an average value of 3 080 000 pounds per square inch with concrete aged 16 months, its variability being somewhat greater than that of the compressive strength. This variability in strength and elastic properties appears to have had little influence on the fire resistance of the concrete.

(2) DEFORMATION AND TEMPERATURE.—Maximum unit expansion of 0.0023 to 0.0046 were observed in fire tests of reinforced-concrete columns, the average being about one-half of that found for cast-iron columns. The indications are that a considerable portion of the expansion was due to the vertical steel reinforcement, maximum expansion being coincident with temperatures in the reinforcement of 400 to 500° C, in which region yielding of the metal takes place under stresses that were likely to be induced in the bars by their higher expansion rate with reference to the concrete. With limestone or trap-rock concrete and with lateral ties not less than 12 inches apart no tendency for the vertical bars to buckle before failure was noted.

Following maximum expansion, the columns gradually compressed, the rate being less rapid than for steel and cast-iron columns (Figs. 169 to 171). Temperatures at failure in the center of the trap-rock concrete columns that failed in the fire test averaged 450° C (842° F), and in the vertical reinforcing bars 894° C (1641° F).

(3) CHARACTER OF FAILURE.—The columns failed locally by compression or by combined compression and shearing on inclined planes, the vertical bars buckling and lateral ties or hooping, breaking, or yielding at the point of failure. No large lateral deflections developed either before or at failure.

(*e*) TIMBER COLUMNS

(1) QUALITY OF MATERIAL.—The timber was long-leaf pine and Douglas fir of select structural grade and conformed with the requirements of published specifications for structural timber (Table 2).

(2) DEFORMATION AND TEMPERATURE.—Except for slight expansions noted during the first few minutes of the test, the timber contracted or compressed under the combined load and fire condition, most of the deformation occurring in the wood at its bearings on the metal cap introduced near the top of the column, the crushing of the wood at these points causing progressive depression of the top of the timber columns (Fig. 47, p. 126).

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The temperature in the timber away from the surface and the cap bearings was retarded by the evaporation of moisture and the low heat conductivity of the wood and did not exceed 100° C (212° F) until near failure (Figs. 140 and 141). In the metal cap at the edge of the column bearing the temperatures at failure varied for the different tests from 432 to 544° C (810 to 1011° F) (Table 43, p. 122).

(3) CAUSE OF FAILURE.—The cause of failure of the timber columns was loss of strength of the wood at the cap bearings, due to conduction of heat from the flanges of the metal caps to the bearing plates and into the wood. The consequent softening of the wood caused the columns to slip laterally on their bearings with the steel-plate caps and to fracture the cast-iron caps. While the test fire reduced the area of the columns by 29 to 55 per cent their resistance to fire and load outside of the bearings was not fully developed in the tests.

2. USEFUL LIMITS

The useful limit point with reference to fire exposure of columns is here taken as the limit of the period within which the effects on the elements designed to carry load do not permanently impair their essential structural properties to such extent as to make them unsuitable for further use.

(a) STRUCTURAL-STEEL AND CAST-IRON COLUMNS

The test characteristics indicate that for columns restrained at the ends and with slenderness ratio not exceeding 80, the useful limit is approximately equal to the period of expansion as given in Table 43 (p. 122) and Fig. 45. The measured center deflections of columns within the given $\frac{l}{r}$ limit did not exceed ½ inch at the end of the expansion period. The deflection of some of the columns of higher slenderness ratio was larger, although at the given stage all measured deflections were within 1 inch. Since failure in details such as riveting was not apparent in any test, unfavorable conditions in this respect could not have existed at any time preceding failure.

Recent tests made by the Bureau of Standards on specimens of structural shapes that were heated under load up to the point of maximum expansion and cooled indicate little or no distortion of shape, loss in strength, or change in elastic properties. The compressive set on cooling and removal of load was not large enough to be objectionable as applied to a building column.

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In the case of the structural-steel columns with 2-inch Chicago limestone or Joliet gravel concrete coverings the interval between maximum expansion and failure was equal to 73 per cent, or more, of the expansion period, and, considering the consequent lower rate of deformation of the column, the useful limit may possibly extend considerably beyond the expansion period. The same applies to the columns with 4-inch concrete coverings made with the Chicago limestone aggregate, which withstood the fire test in excess of 8 hours.

Damage to the covering is not considered in the above discussion, as it is commonly not regarded as a load-carrying element.

(*b*) PIPE COLUMNS

For the unreinforced pipe column the useful limit can be taken as the period of expansion in the fire test.

For the reinforced pipe column the transfer of load, as caused by successive heating and expansion, first to the pipe and later to the structural steel reinforcement, confounds the relation existing between maximum expansion and useful limit. The temperatures on the outside of the pipe indicate that the useful limit was reached after between 20 and 30 minutes of fire exposure. The center lateral deflection at the end of this interval was ¾ of an inch.

(*c*) REINFORCED-CONCRETE COLUMNS

The reinforced-concrete columns expanded less than the steel and cast-iron columns, reached their point of maximum expansion at a relatively earlier stage, and the subsequent compressive deformations and center deflections were smaller. Comparisons of the temperatures of the reinforcing bars and the attending unit deformation indicate the presence of high compressive stresses, although with concrete of the given aggregates and the methods of tying employed no tendency to buckle before failure was noted. For the columns that withstood the 8-hour fire test and sustained large additional loads before failure it is believed that the useful limit extended up to the end of the fire test. In the case of the columns that failed in the fire test or sustained but small additional load after 8 hours the matter is more indeterminate, although it can be stated that large compressive deformations did not obtain until within the 1½ hours preceding failure.

These considerations do not preclude damage to the outer concrete covering which it might be necessary to repair or replace after fire exposures within the useful limit.

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(d) TIMBER COLUMNS

The effect that limits the usefulness of timber columns after fire exposure, as far as it concerns the timber, is surface damage from burning away of the wood, with consequent reduction of effective section. Large compressive deformation of the column due to impairment of the unburned wood from the heat apparently obtains only for points nearer failure. The amount of reduction in area that can be allowed depends on the basis of design. In common with general practice, the whole area of the test columns was assumed to carry load, no portion being deducted as protection. Assuming that the initial factor of safety was no more than adequate, minor reductions in area would render the column unfit for further use. Practical requirements relative to minimum and standard sizes often result in loadings lower than the safe working limit, in which case proportionate reductions in area due to fire could take place without impairing the safe load-carrying capacity.

With reference to fire effects on unprotected steel and cast-iron bearings, the rapid rates of deformation obtaining after 20 minutes in tests of unprotected timber columns indicate that the useful limit in this particular did not extend much beyond this period (Fig. 47, p. 126). For the protected timber columns the useful limit in this respect extended to within about the same time interval (15 to 30 minutes) before failure, as in the tests of unprotected timber columns.

(e) PRACTICAL APPLICATION

The above conclusions relative to useful limits find application where it is desired to design and protect columns so they will not be materially injured by a fire that consumes the contents of the structure supported by them. In applying the results of these tests for this purpose it is necessary to make deduction for variations in material and workmanship not developed in the tests. The percentage reduction can be safely taken as that used in deducing ultimate fire resistance periods in the concluding section of this paper. The period of expansion is influenced to less extent by incidental conditions, such as load-carrying capacity of covering materials, than is the time to failure.

While the useful limit may be made the basis of design of columns and coverings in special cases, it is believed that the most general use of the test results will be that based on the ultimate fire-resisting point; hence no detailed application of the former will here be made.

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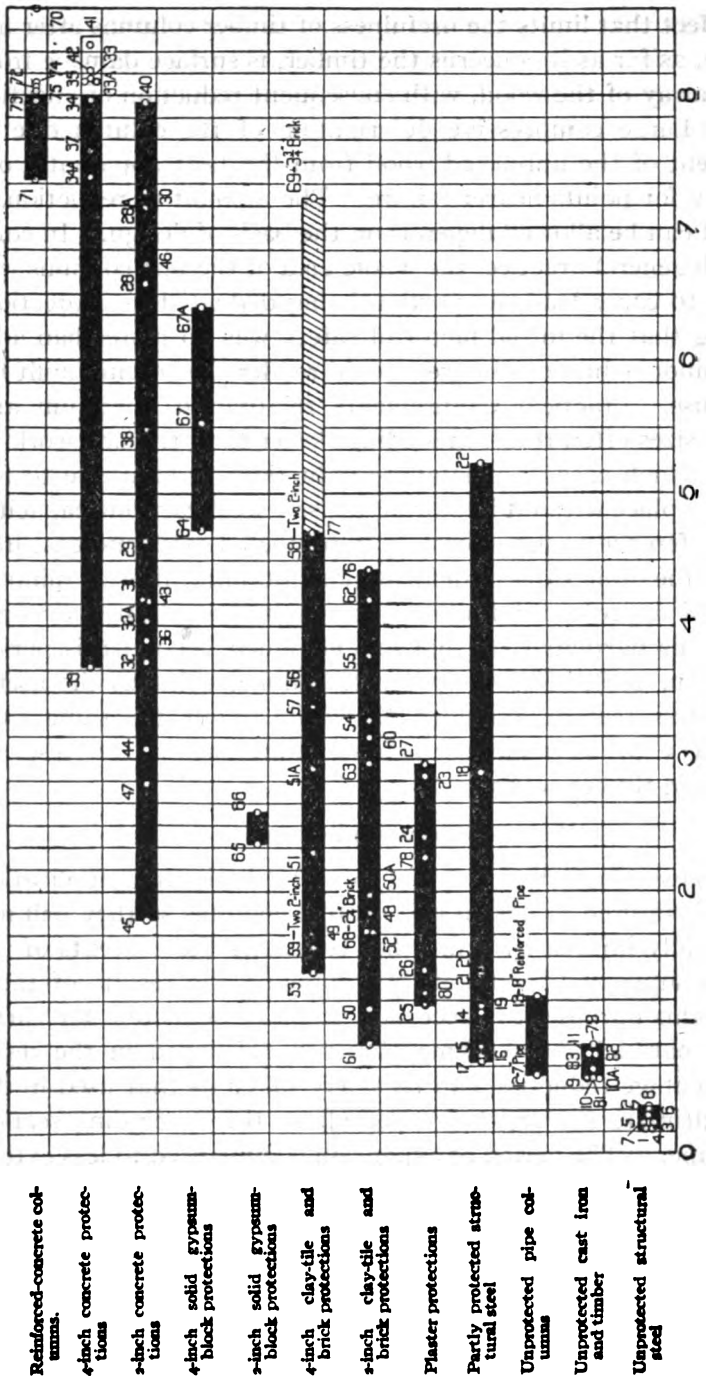

FIG. 48.—*Results of fire tests compared by groups*

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3. DISCUSSION OF TEST DATA

The considerations herewith given relate to test duration, period of expansion, and the causes immediately affecting them. Comparisons of the time to failure of columns in the fire-test series arranged by groups are given in Fig. 48, which also provides a measure of the variation obtaining within each group. Further comparisons of time to failure and period of expansion are given in Table 43 (p. 122) and Figs. 42 (p. 114) and 45 (p. 121). In the discussion the time periods will be given to the nearest minute.

(a) DIFFERENCE IN COLUMNS, TEST EXPOSURE, AND SERVICE CONDITIONS

(1) VARIATIONS DUE TO DIFFERENCE IN COLUMNS.—Among the effects tending to produce variation in results with given types of protection for the same thickness of covering are size and shape of structural section and consequent differences in intensity of loading and resistance to fire; differences in fire-resisting properties of subclasses of covering material within each group, such as concrete of the different aggregate combinations and hollow clay tile of the various types of clay; variations in methods and details of application; incidental variations in material and workmanship. These are discussed in connection with each group whenever the test results developed information on any given particular.

A fairly wide range of structural section was employed in the tests, and it is believed that the influence of this factor was sufficiently determined.

One or more representative materials from each of the main subclasses of covering material were introduced. It is appreciated that considerable differences in mineral composition and structure may exist between material in a given subclass as occurring in different localities, and, while the difference in fire-resisting properties may be of minor order, wide application of the results of the tests will have to be made with care until the knowledge on the subject has been extended by further tests.

The full range in results due to difference in methods of application and in incidental variations in materials and workmanship was not developed in the tests, as this required a greater number of test duplications than could be introduced. In general, poured coverings are subject to smaller incidental differences than those built

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up from small units; also, if stability is assured, a large element of variability is eliminated. It is also thought probable that columns and protections in buildings are subject to greater variability than those constructed for the tests, the latter representing workmanship of more uniform quality.

(2) VARIATIONS DUE TO DIFFERENCE IN LOAD AND FIRE CONDITIONS.—In the tests the amount of load applied on a given column section or column type was subject to minor variations only, the errors being generally within 1 per cent. Comparisons of the initial deformations produced in opposite flanges of individual columns indicate maximum outer-fiber stresses in several tests up to 25 per cent above what would obtain for uniform distribution of load. These can be ascribed mainly to bending stresses induced by uneven bearings and are no larger than those generally incident with compression tests or with normal loading of columns in buildings.

The loads applied in the tests were about 10 per cent higher than those derived from the column formula employed. While floors in buildings are subject to overload by more than this amount, the resultant load on the columns is likely to be reduced by the distribution of load concentrations to several columns and by the equalizing effect of loadings from several stories.

As exposed to fire in buildings, individual columns may take added load due to higher temperature, and consequent greater expansion, than the adjacent columns of the story, particularly if they are not fully protected. For steel or cast-iron columns the additional load assumed is limited by the ultimate flexural strength of their connections to the floor beams framing into them. In the case of interior columns the additional load possible to assume on this basis will seldom exceed 20 per cent of the nominal floor loads. Reinforced-concrete columns have more rigid floor connections than steel or cast-iron columns, but, due to their lower expansion and rate of temperature rise, more unfavorable loading conditions as due to unequal expansion are not likely to occur.

The furnace exposures are given in the test results as percentages of the average of all tests, the quantity compared being the area under the average time-temperature curve of each test. For the shorter tests with upper limit of duration of about one hour the difference in exposure may have had a considerable influence on results, considering also that the percentages given do not indicate

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the full measure of the difference due to lag of the pyrometers. For the longer tests, variations in test results due to difference in furnace exposure were apparently of minor importance.

Comparing the average time-temperature curve of the column tests with the reference curve (Fig. 39) a fair agreement is seen to have been attained. The reference curve was adopted as being consistent with furnace exposures used in previous fire tests, and there was at the time no distinct understanding whether it should represent indicated temperatures or temperatures corrected for lag and radiation effects, little information on the extent of these effects being available. Investigations conducted subsequent to the completion of the column tests disclosed large effects due to lag during the initial period of the fire exposure and smaller, but more persistent, effects due to radiation (p. 101–106, Figs. 40 and 41). Any time temperature relation adopted as standard is necessarily more or less arbitrary, particularly for the initial period. Temperatures attained in fires afford some guidance, although they indicate a wide range in intensity. From the information available, points on the reference curve, considered either as indicated or as corrected temperatures, while not the maximum attained in fires of exceptional intensity, represent severe fire conditions.

(b) UNPROTECTED COLUMNS

Included under this head are structural steel, cast-iron, and pipe columns that were tested without protective coverings, all parts of their sections being assumed to carry proportionate portions of the applied load.

(1) STRUCTURAL-STEEL.—The time to failure of the unprotected structural-steel columns varied from 11 to 21 minutes (Table 42*a*, p. 108). The difference in results for the various column types, while due in part to variation in furnace exposure, is attributable also to difference in thickness of metal and in the unit loads sustained, sections with thin members under the higher unit loads failing sooner than sections whose members were arranged to form heavy metal thickness. Fig. 49 is a plot between unit load sustained, and time to failure in tests of unprotected structural-steel columns, showing the inverse relation obtaining between them. The variation in the unit load applied was due to difference in slenderness ratio between the different sections.

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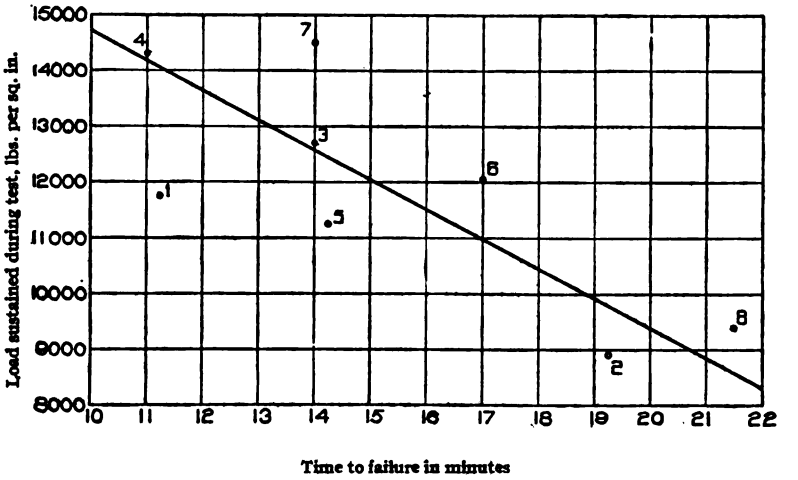

FIG. 49.—*Effect of load on fire resistance, unprotected structural-steel columns*

The average time to failure of the eight structural-steel sections was 15 minutes and the average period of expansion was 13 minutes (Table 43). Maximum temperatures from 578 to 668° C (1072 to 1234° F) were attained on the outside of the metal near the edges. It is difficult to estimate the average effective temperature, since the rise was too rapid to permit assuming temperature uniformity over the thickness of the metal. Tests recently made by the Bureau of Standards on small specimens indicate that for the loads sustained by the unprotected columns the failure temperatures of the structural steel fell within the limits 550 to 650 ° C (1022 to 1202° F).

(2) CAST-IRON.—The average time to failure of the unprotected and unfilled cast-iron columns, one of which was tested with ends restrained and two with unrestrained ends, was 34 minutes, the variation being less than one minute for the three tests. The periods of expansion varied from 22 to 24 minutes. The longer test periods and higher temperatures attained as compared with the structural-steel columns can be attributed largely to the lower allowable unit loads applied to cast iron and the relatively smaller surface exposed to the fire. As judged by the direction of the lateral deflection, failure in compression appears to have started on the side having the thinnest metal. Filling the interior with concrete (test No. 11) increased the time to failure 11 minutes, with a smaller proportionate increase in the length of the expansion period.

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In the two fire and water tests of cast-iron columns, water was applied when the columns had attained maximum expansion, the metal being at low red heat. No cracks developed in the metal, the only effect being permanent lateral deflections of, respectively, 3/8 and 7/8 inch toward the side on which water was applied.

(3) PIPE COLUMNS.—The time to failure of the 7-inch pipe column was 36 minutes and maximum expansion was attained at 14 minutes.

The 8-inch reinforced pipe column failed after 1 hour 12 minutes, and maximum expansion was attained at 52 minutes. The column expanded only a small amount after 25 minutes, the total expansion being about 1/4 of an inch. The high temperatures on the surface of the pipe indicate that during the last half of the test period nost of the load was carried by the structural-steel reinforcement.

(c) PARTLY PROTECTED COLUMNS

Filling the reentrant portions or interior of the structural-steel sections with concrete greatly increased their fire-resisting property as compared with that in the unprotected condition (Table 42*b*). The difference was due to slower temperature rise in the metal resulting from the heat insulating and absorbing properties of the filling and to the load-carrying capacity of the concrete, the longer time intervals between maximum expansion and failure being due in great part to the latter factor.

Temperature differences up to 450° C (810° F) existed between the exposed flanges and the protected webs, the temperature of the latter at failure being generally below 500° C (932° F).

(1) EFFECT OF SECTION AND SIZE.—No decided effect due to variation in shape of structural section was noted except as it affected the size of the column.

In Fig. 50 is shown the relation between time to failure and area of material in cross section for all tests of partly protected columns, except No. 22, which was eliminated as being in effect a protected column. The relation shown accounts for the variation in results within the group as being due in large part to difference in size.

(2) EFFECT OF CONCRETE AGGREGATE AND TIES.—In the tests of short duration the concrete aggregate appears to have affected

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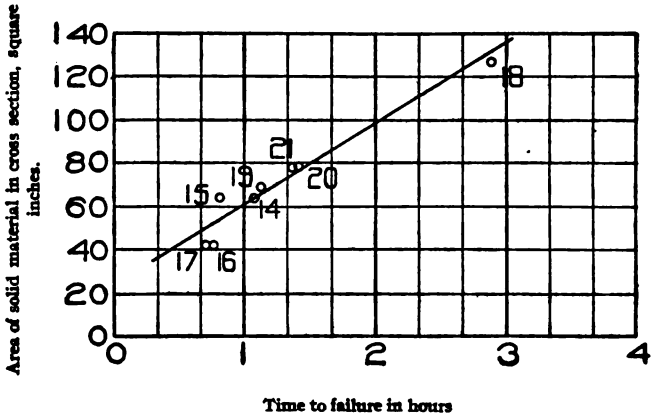

FIG. 50.—*Effect of size, partly protected columns*

results to a minor extent only, while in those of longer duration (Nos. 18 and 22) the results compare with those obtained for concrete protections, which they more nearly resemble.

Metal ties were placed in the concrete of all partly protected columns where not contained by the section members as in the latticed columns. No tendency was noted on the part of the filling to spall, buckle, or otherwise come loose before failure. The extent to which the ties functioned to prevent such effects is not fully determinate, although in tests of concrete protections made with the same aggregate little cracking or spalling was noted within the given time periods.

(*d*) PLASTER ON METAL-LATH PROTECTIONS

(1) MATERIAL AND DESIGN.—The Portland-cement plaster, of proportion 1 : $\frac{1}{10}$: 2½ volume parts of Portland cement, hydrated lime, and coarse lake sand, as mixed by the workmen for the protections and tested in 2-inch cubes, developed an average compressive strength of 1677 pounds per square inch at 28 days and 2623 pounds per square inch at an average age of 16½ months, with maximum range in individual tests of 44 per cent below and 60 per cent above the given averages (Table 25, Fig. 23). Cubes made in the laboratory of the same materials and average water content (16.7 per cent) gave lower compressive strength, both as stored in air and in water (Table 28, p. 356).

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The metal lath was wrapped around the structural-steel columns on bars or on pressed-steel channels acting as spacers, and the plaster applied in layers ⅞ to 1⅛ inches thick, each consisting of two body coats. The double-layer protections had a ¾-inch air space between layers. For the cast-iron column the plaster was applied on high-ribbed metal lath supported directly on the column. A broken air space of about ½ an inch thickness was formed next to the metal, due to failure of the plaster to fully fill the space back of the lath.

(2) TEST RESULTS.—The average time to failure of the structural-steel columns with single layer protection was 1 hour 16 minutes, and of those with double-layer protection 2 hours 38 minutes, the two tests of each varying from the average by less than 15 minutes (Table 42*c*, p. 109). The cast-iron column protected by a single layer of 1½-inch average thickness with a broken air space between it and the iron (test No. 27) stood up longer by a few minutes than any of the other columns in the group.

The temperature distribution across the section was generally very uniform, and the variations in the length of the column were not large (Figs. 97 and 98). This was due to the air space between the covering and the structural section, which permitted free heat interchange in the column, unmodified by the temperature gradients in the covering material.

(3) CRACKING DUE TO EXPANSION OF COVERING.—During the first 20-minute period in all tests of Portland-cement plaster protections, cracking and disruption of the plaster took place below the bracket near the top of the column. This was evidently due to expansion of the plaster layer, which was restrained at the top of the column and at the bottom bearing. This effect appears to have had little influence on the time to failure in the given tests, the region of maximum column temperature and failure being in all cases within the middle 4 feet of the column height.

The sand used in the plaster was high in insolubles (chiefly silica) and low in calcite and dolomite (Table 14, p. 342).

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(4) EFFECT OF VARIATION IN DETAILS OF APPLICATION.—Comparing the layer thickness of test No. 23, where the plaster was applied on expanded metal lath, with that of test No. 24, where woven-wire lath was used, and also the outer layer thickness with the inner in test No. 110, applied, respectively, on expanded metal and on woven wire, a heavier layer thickness by ⅛ inch was attained in all cases with the expanded metal (Tables 3*d* and 4*d*). This difference in layer thickness may account in part for the longer test duration of No. 23 as compared with No. 24. Further indications that layer thickness is an important element in the protection given is had in the case of tests Nos. 25 and 26, where, with a difference in layer thickness of ⅛ inch, a difference in time to failure of 16 minutes obtains.

The tests developed no evidence that the method of supporting the lath had any influence on results (cf. Sec. IV, par. 2*a*, p. 56).

All double coverings had an air space about ¾ of an inch wide between the inner and outer layers. No tests were made with a single layer equivalent in thickness to that of two double layers, hence no direct evidence relative to the value of the air space as an insulating medium was obtained.

(5) EFFECT OF WATER APPLICATION.—The water carried away some loose pieces near the top of the column where cracking and spalling had taken place during the fire period (Fig. 87). It exposed the lath at the corners of the outer layer for portions of the height. The test did not appear to have materially injured the insulating value of the covering since the time to failure in the subsequent fire test nearly equaled that of the corresponding test (No. 23) in the fire series (Table 44, test No. 110).

(*e*) CONCRETE PROTECTIONS

(1) MECHANICAL PROPERTIES OF THE CONCRETE.—The compressive strength of 8 by 16 inch cylinders made from 1:2:4 gravel or crushed-stone concrete mixed for the column coverings under conditions approximating those of building practice, averaged 1520 pounds per square inch at 29 days and 2100 pounds per square inch at an average age of 15 months, the maximum range of individual test results being 118 per cent above and 57 per cent below the given average values (Table 21, p. 346). As shown in Fig. 15 (p. 66), the range in results increased with the number of tests in the group, indicating that in the groups with the smaller number of tests the full possibility of variation was not developed.

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The principal cause of the variation in strength was apparently difference in consistency of the concrete mixtures. Increasing the time of mixing from one minute to two minutes gave an indicated increase in the average compressive strength of 45 per cent (Fig. 19, p. 69).

The modulus of elasticity of the concrete varied approximately with the compressive strength from less than 1 000 000 to over 4 000 000 pounds per square inch (Fig. 21). This large variability in the mechanical properties of the concrete appears to have had little influence on its fire-resistive properties, the latter depending chiefly on the mineral composition of the aggregates employed.

(2) FUNCTION OF CONCRETE AS A COVERING MATERIAL.—Concrete applied as a protective covering or filling to steel or cast-iron columns retards the temperature rise in the metal when the column is exposed to fire and further retards the failure by carrying portions of the column load proportionate to its relative area and rigidity as compared with the metal.

The protections were applied as square or round coverings, generally 2 inches and 4 inches in thickness as measured from the surface of the covering to the metal. The time to failure in the fire tests varied from 1 hour 47 minutes to 7 hours 57 minutes for the 2-inch protections, and from 3 hours 41 minutes to over 8 hours for the 4-inch protections (Table 42*d*, p. 110).

(3) VARIATIONS DUE TO CONCRETE AGGREGATE.—With a given thickness or size of covering the main cause of variation in results was the difference in fire-resisting properties of concrete made with different aggregates. In this particular the concrete can be placed in three groups. That giving the most unfavorable results was the concrete made with Meramec River sand and gravel, a number of large cracks forming early in the tests followed by spalling of large and small pieces of concrete not held by the ties (test Nos. 39 and 45). This sand and gravel consist almost wholly of quartz and chert grains and pebbles, the gravel having a particularly high chert content. Both minerals are forms of silica (SiO_2), the quartz being crystalline and anhydrous, and the chert amorphous, with a variable amount of water in chemical combination. On being heated part of the combined water in the chert is liberated, and the consequent vaporization disrupts the pebbles. Other causes of disruption of concrete made with

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siliceous aggregates are abrupt volume changes, points of which are known to exist for chert as low as 210° C (410° F). Quartz has a decided point of abrupt volume change at 573° C (1063° F), where it is transformed into the mineral tridymite, the change extending over a considerable temperature range when the heating is rapid. Liquid inclusions contained in small cavities formed when the rock crystallized from the molten condition may be the cause of some of the cracking incident with fire exposure.

The middle group includes concrete made with trap rock, granite, sandstone, and hard-coal cinder.

In tests with trap-rock and cinder concrete a small amount of cracking developed during the last part of the fire period, but no spalling of note occurred before failure. In the granite-concrete protections the cracks developed earlier in the test, and portions of the corners spalled off during the last 30 minutes of the test period. In the tests with sandstone-concrete protections cracking and spalling of corners outside of the wire tie began in the first 30-minute period and continued during the next hour, after which there was little apparent change before failure. The spalling exposed portions of the flange edges, which to some extent hastened the failure. The average time to failure in tests with sandstone-concrete protections was intermediate between those with trap-rock and those with cinder concrete. The cracking of sandstone concrete after a short fire exposure can be ascribed mainly to the abrupt volume change of the constituent quartz grains as noted above.

Fusion of the trap-rock concrete occurred where the tests extended beyond 7 hours, the concrete being affected to a depth of about 1½ inches. Flowing of concrete due to fusion, while not general, occasionally formed pockets up to a 2-inch depth. Incipient fusion to about the same depth occurred in the 4-inch granite-concrete protections, although no actual flowing of concrete took place.

The third group comprises protections of Chicago-limestone concrete and Joilet-gravel concrete. The composition of this gravel is similar to that of the Chicago limestone, and the fire-resisting properties of the concrete made with each compare quite closely. Very little cracking resulted on exposure to fire and their heat-insulating value was increased by the change of the calcium and magnesium carbonate to the corresponding oxides.

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This process retarded the flow of heat through the region of change and left material of good insulating properties. Immediately after test the surface of the concrete was firm, but after a few weeks exposure the hydration of the oxides caused slaking and crumbling of the calcined material (Fig. 62, p, 221).

(4) COMPARISON OF 2 AND 4 INCH PROTECTIONS.—In the comparison given in Fig. 51, the tests of concrete protections are arranged by groups as defined in the preceding paragraph, the line in the case of the middle group connecting the average value for each thickness. Test Nos. 40, 46, and 47 are omitted in this comparison on account of extreme shape and size of section, and No. 44 on account of leaner concrete mixture.

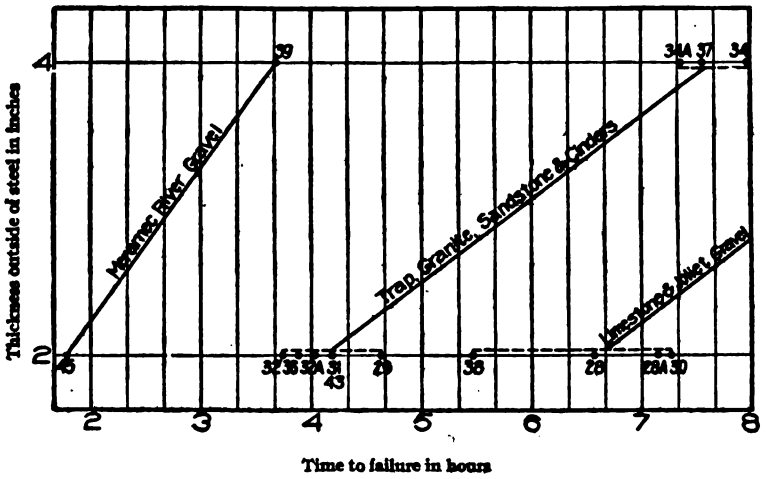

FIG. 51.—*Comparison of 2-inch and 4-inch concrete protections*

The time to failure under working load was not determined for the 4-inch limestone-concrete protections, as they were loaded to failure after withstanding the fire test in excess of 8 hours. They all attained maximum expansion within the last 30 minutes of the 8-hour fire period, and from comparison with results obtained with the corresponding 2-inch protections, failure in the case of the 4-inch protections would not have taken place before the end of 10 hours, assuming the same load and the same furnace temperature rise as obtained during the 8-hour period.

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(5) EFFECT OF SIZE.—In Fig. 52 the time to failure in tests with concrete protections of the middle group is plotted against the area of steel and concrete in the cross section. Variations

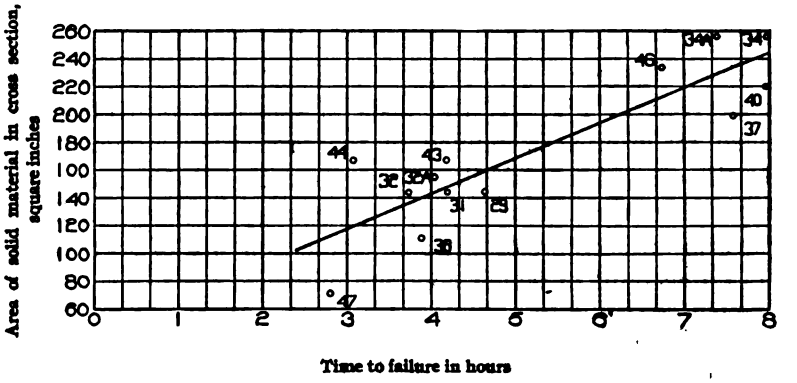

FIG. 52.—*Effect of size, concrete protections*

from the general trend can be accounted for as due to concrete aggregate, proportions of concrete mixture, type of section, and to incidental differences in test conditions and test columns. The extreme variation in furnace exposure as measured by the area under the furnace temperature curves is within 2 per cent as between all the tests plotted, except Nos. 34 and 34*A*, between which there is a difference of 4 per cent (Table 42*d*), which latter may be responsible for the difference in failure time of the two tests.

(6) EFFECT OF STRENGTH OF CONCRETE.—Some decrease in fire resistance due to leaner mixture may be noted by comparison of Nos. 33 and 33*A* with No. 35, and No. 43 with No. 44.

No evidence was developed that variation in the strength of the concrete of the same aggregate and proportion of mixture had any appreciable influence on the results of fire tests of concrete protections. This was due to the large change in mechanical properties produced by the heat. Concrete as made with different aggregates preserves strength to different degrees on exposure to fire. This had a decided influence on results, the longer test periods, and particularly the longer intervals between maximum expansion and failure of the limestone concrete and Joilet-gravel concrete coverings, can be attributed in a great part to this cause.

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(7) INFLUENCE OF SHAPE OF SECTION AND COVERING.—As in the case of the partly protected columns, the only well-defined effect of change in shape of structural section was primarily due to the resulting difference in the cross-sectional area of the covered column.

Round and square coverings of trap-rock concrete displayed only minor differences in the number and size of cracks that developed before failure. Fine vertical cracks 2 to 4 inches from the corners formed in the square coverings at somewhat earlier periods than the first cracks noted in the round coverings. In neither case do these cracks appear to have had any appreciable influence on the time to failure. Coverings made of concrete more subject to cracking may possibly develop greater differences due to shape, although with concrete made with highly siliceous aggregates, the influence of the aggregate is so large that other effects are small in comparison. This is shown in test No. 45, where the round, siliceous-gravel concrete covering sustained severe cracking and spalling early in the test which caused failure over one hour earlier than in any other test of concrete protection.

(8) FUNCTION OF THE WIRE TIE.—In all tests of concrete protection, except Nos. 28A, 33A, and 47, the concrete was tied by a wire tie wound spirally around the structural-steel section. In Nos. 28A and 33A the concrete aggregate was Chicago limestone. No cracking of consequence developed before the end of these tests, and the absence of the tie had no influence on the results. In the case of the cinder-concrete protection in test No. 47 no cracking of note occurred until near failure and after the column had sustained large compressive deformation. About two minutes before failure most of the covering fell off. This would have been prevented if the wire tie had been present, although the temperature of the metal and the deformation of the column was such that failure was imminent.

It is not possible to state the extent to which the tie functioned in all tests of concrete protection, but it was undoubtedly of value where there was any tendency for the concrete to crack and fall off before failure. In the case of Meramec River gravel concrete and sandstone concrete, spalling of portions of the covering outside of the tie occurred early in the test, the concrete on the middle of the flanges and webs being apparently held by the ties.

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(9) EFFECT OF WATER APPLICATION.—In the three tests where the wire was placed in the coverings the water pitted the exposed faces of the concrete and carried away portions of the corners and sides, leaving parts of the flanges and flange edges exposed. The damage was most marked in the regions where cracking was noted during the fire period (Figs. 83 and 84, p. 242–243).

In the case of the one covering that was not tied the water loosened or carried away most of the protection on the flanges, leaving the column in the condition of partial protection. Most of the damage was incurred after 2 minutes of water application, the total period being 5 minutes.

(*f*) HOLLOW CLAY-TILE PROTECTIONS

(1) MECHANICAL PROPERTIES OF THE TILE.—The average compressive strength of specimens of the hollow clay partition tile used in the column coverings was 5350 pounds per square inch as tested on end and 4370 pounds per square inch tested on edge, the maximum variation above these values being 134 per cent and below 78 per cent, of the lower average value. For the same type of clay the range in results was smaller. The strength was generally proportional to the density of the tile as indicated by percentage of porosity and of absorption (Table 31, p. 358–359).

In transverse tests of hollow tile the average computed outer fiber stress at failure was 527 pounds per square inch and the shear 233 pounds per square inch, the failure being apparently due to combined shear and tension. The range in results was larger than in the compression tests (Table 32).

There appears to be little relation between the mechanical strength and the fire-resistive properties of the tile, the latter depending mainly on the type of clay. This is significant as specifications based on the mechanical properties often disqualify tile desirable from the standpoint of resistance to fire.

(2) TEST RESULTS.—In tests of hollow-clay tile protections, using tile of the given types of clay applied according to the methods previously described, the time to failure ranged from 50 minutes to 4 hours 42 minutes. This large range in results was due to a number of factors that influence the effectiveness of this type of protection, including, besides type of clay, methods of manufacture of the tile, thickness of shells and webs, the presence or absence of concrete or other filling, and the methods used for tying the tile (Table 42*e*, p. 111).

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(3) VARIATIONS DUE TO TYPE OF CLAY AND DETAILS OF MANUFACTURE.—The extent to which the tile cracked and spalled on exposure to fire varied with the type of clay of which it was made and the degree of hardness to which it was burned. All of the tile used in the coverings was straight, nonporous partition tile, burned without sawdust or other filling, except the round tile in test Nos. 62 and 63, which was porous.

The semifire-clay tile generally gave the most favorable results in the fire tests, and of the two represented the tile of medium hardness and having heavier webs and shells developed few cracks and little spalling before failure. In the coverings of round, porous semifire-clay tile a number of vertical cracks formed after a short fire exposure, which became wider as failure was approached. Few other disruptive effects were noted, almost all material remaining in place till the end of the test. Only minor differences in behavior were noted between tile made of the two kinds of surface clay, cracking and spalling of outer shells and buckling out of the tile being characteristic of tests of both. In the case of shale tile, these effects were even more pronounced, severe cracking taking place during the first few minutes of the test, followed by general spalling of outer shells.

(4) COMPARISON OF 2 AND 4 INCH PROTECTIONS.—In Fig. 53 is given a comparison of time to failure in tests with 2 and 4 inch

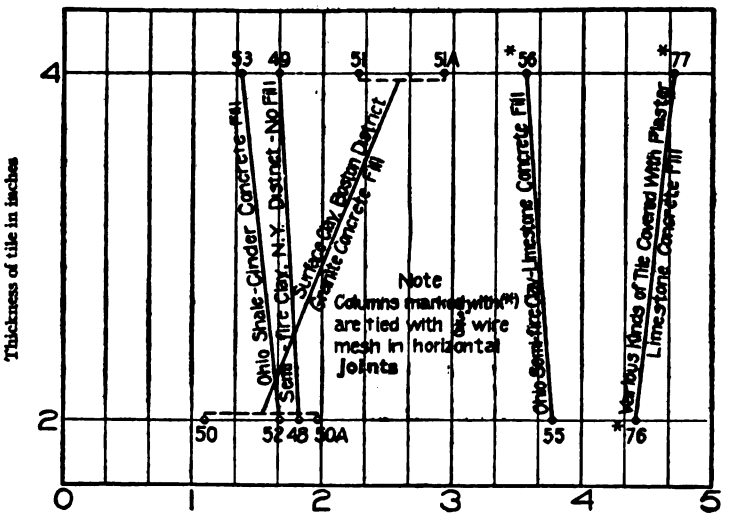

FIG. 53.—*Comparison of 2-inch and 4-inch hollow clay-tile protections*

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hollow clay-tile protections, all other details being comparable except as noted. The tests show little difference between the two, the thickness of the air space and minor variations in thickness of shells having apparently little influence on results. The difference in results in test Nos. 50 and 50A as compared with 51 and 51A can be attributed to larger differences in thickness of shells and webs and also to the greater stability of the 4-inch tile set on end and tied with outside wire ties, as against the 2-inch tile laid flat in 6-inch courses without ties (p. 137-138).

(5) EFFECT OF SIZE.—In Fig. 54 is shown the relation between the time to failure in all tests of hollow clay-tile protections made with nonporous partition tile and the total area of solid material

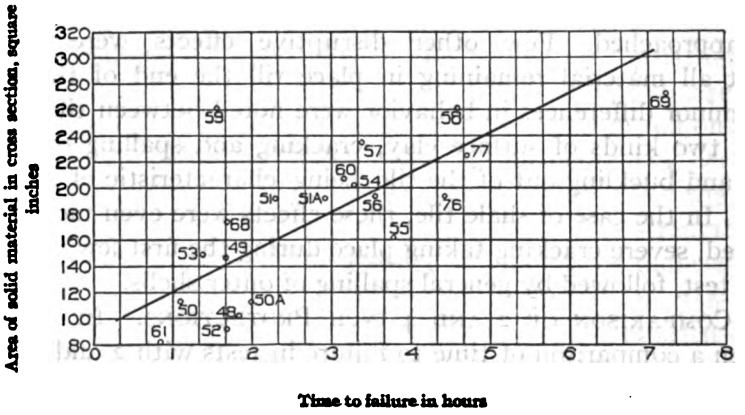

FIG. 54.—*Effect of size, clay-tile and brick protections*

in the cross section, a fairly consistent variation of the one with the other being evident. Nos. 58 and 59, having double layer of tile with tile filling, gave less favorable results in this comparison than tests of protections with a single layer of tile and concrete fill.

(6) EFFECT OF TIES AND FILLING.—Fig. 55 gives a comparison of results attained with the two methods used for tying the tile and of including or omitting the concrete or tile filling. As shown on the diagram and confirmed by test characteristics, the mesh in the horizontal joints is a little more effective in holding the tile than the outside wire ties; also the concrete or tile filling is an important element in the protecting property of a hollow-tile protection.

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The efficiency of wire mesh in the joints as against outside wire ties, in the case of protections of surface-clay tile with hollow tile filling, is shown by comparison of results in test Nos. 58 and 59. It should be considered in this connection that minor parts of the difference may be due to incidental variations in workmanship and test conditions, and also that protections of tile less subject to cracking and disruption on exposure to heat would show relatively smaller differences in results, due to the methods used for tying the tile.

The concrete filling not only serves as a protecting medium, but also assists in holding the tile in place by adhesion. In test

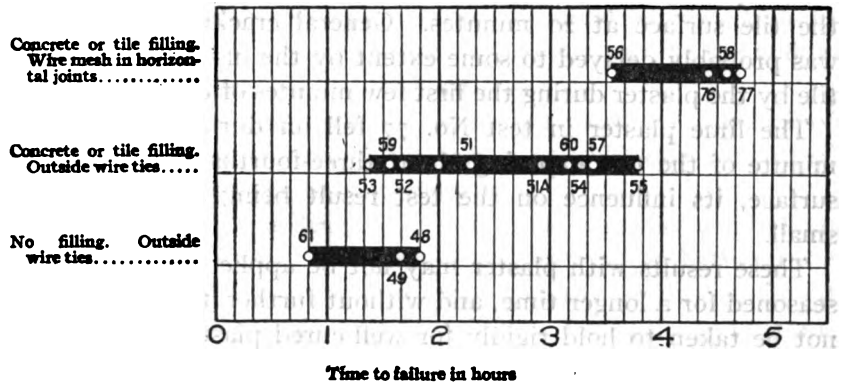

FIG. 55.—*Effect of ties and filling, hollow clay-tile protections*

No. 60 the concrete filling was placed before the tile was set, the tile being bonded to the filling with a thin mortar joint and tied with outside wire ties. A large number of tile units fell off early in the test, the behavior in this particular being distinctly different from that of protections with concrete fill placed after the tile was set, where up to points near failure the inner shell generally remained in place.

As indicated by the average temperatures in the steel at failure, the concrete filling carried portions of the applied load varying with the area, the higher temperatures at failure being generally

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incident with the tests having the larger fills. In the latter tests the periods between maximum expansion and failure were comparable with those obtaining for concrete protections, whereas in the tests where the concrete fill was omitted or of small area this time interval was relatively short (Table 43, p. 122).

(7) EFFECTIVENESS OF PLASTERING.—The tile in test Nos. 76 and 77 were covered with standard applications of gypsum and of lime plaster, respectively, applied 3 days after the concrete fill was placed, the columns being tested 42 and 45 days after plastering.

In test No. 76 the gypsum plaster began to fall off early in the test, exposing a few tile units at 2 minutes and more than half of the tile surface at 20 minutes. General cracking and spalling was probably delayed to some extent by the insulation given the tile by the plaster during the first few minutes of the fire exposure.

The lime plaster in test No. 77 fell off during the first half minute of the test, exposing about three-fourths of the total tile surface, its influence on the test result being apparently very small.

These results with plaster may not be applicable where it has seasoned for a longer time, and without further tests they should not be taken to hold rigidly for well-cured plaster coatings that from conditions of normal exposure are thoroughly dry.

(8) EFFECT OF WATER APPLICATION.—The water generally carried away the tile that had been decidedly damaged during the preceding fire exposure, although adjacent courses of relatively sound tile were in some instances carried down along with those impaired by the fire (Figs. 84 and 85). A large proportion of the effects took place during the first minute. In the test where the tile protection was filled with concrete the condition of the column after the water application approached that of partial concrete protection. In the case of the unfilled columns the steel in the region stripped of tile was unprotected, except for the portions of the mortar joint that adhered to the flanges.

No decided difference was noted between the outside wire ties and the wire mesh in the joints in effectiveness in holding the tile during the water application, although the comparisons were too few to afford definite conclusions.

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(*g*) BRICK PROTECTIONS

(1) PROPERTIES OF THE BRICK.—The common brick used in the brick protections was a wire-end cut brick made in the Chicago (Ill.) district of calcareous surface clay. Compression tests gave average values of 3200, 1960, and 2815 pounds per square inch as tested on end, edge, and side, respectively, and average transverse strength of 862 pounds per square inch, with maximum variations above or below the averages of between 50 and 100 per cent (Tables 35 and 36). The brick was soft, with relatively low fusion point and high percentages of porosity and absorption (Tables 33 and 34, p. 361).

(2) TEST RESULTS.—In the two tests of columns protected by brick approximately the same relation obtained between time to failure and sectional area of the covered columns as for the hollow clay-tile protections (Fig. 54, Nos. 68 and 69).

In No. 68, where the brick was set on edge and end, the lack of stability somewhat shortened the test, a considerable amount of brick falling during the first 30 minutes.

In No. 69, with the brick laid flat, little cracking or spalling developed before failure. Fusion of the brick began between the fourth and fifth test hours, and after test the brick was found fluxed away to a depth of about ½ an inch. The uniform temperature rise in the metal indicates that the fusion of the brick did not contribute greatly to the failure of the column, the same being evidently caused by normal transmission of heat through the covering (Fig. 131, p. 292).

(*h*) GYPSUM-BLOCK PROTECTIONS

(1) STRENGTH AND POROSITY OF THE GYPSUM BLOCK.—Compression tests of the solid gypsum block used for column covering gave an average strength of 468 pounds per square inch, with maximum range in values of 22 per cent below and 40 per cent above the average. The transverse strength averaged 160 pounds per square inch, the extreme range in results of individual tests being a little higher than in the compression tests (Tables 38 and 39). The porosity was quite uniform and high, averaging 62.4 per cent, as based on the total volume (Table 37, p. 363).

(2) COMPARISON OF 2 AND 4 INCH PROTECTIONS.—A comparison in points of time to failure of 2 and 4 inch gypsum protections is given in Fig. 56, where the line connects the average results

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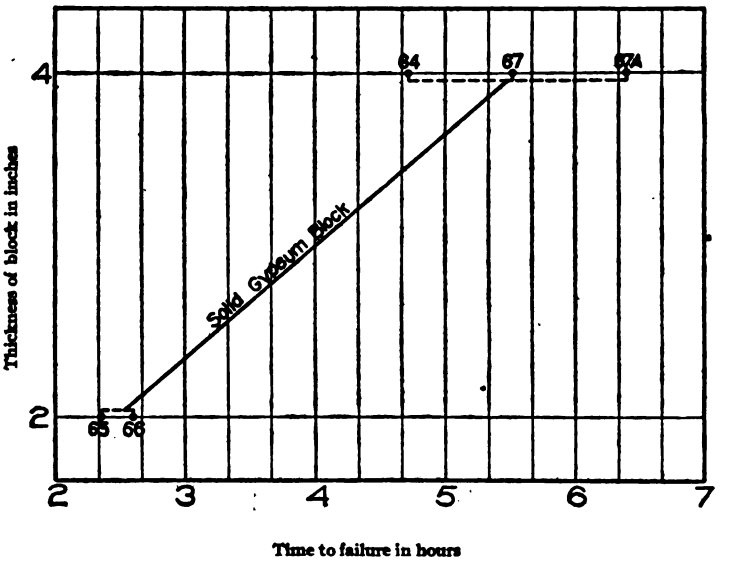

FIG. 56.—*Comparison of 2-inch and 4-inch gypsum block protections*

attained with each thickness. The 2-inch protections withstood the fire test 2 hours 22 minutes and 2 hours 36 minutes, and the 4-inch protections 4 hours 43 minutes, 5 hours 32 minutes, and 6 hours 24 minutes, respectively (Table 42*f*, p. 112.)

The protections were of solid 2 and 4 inch partition blocks set in gypsum and sand mortar, ¾ to 1¼ inch thick between blocks and column flanges, and with metal ties in the horizontal joints, the space between the blocks and the column webs being filled with gypsum block set in place or with a filling poured in place consisting of calcined gypsum, sand, and broken gypsum block.

The variations in results obtaining for each thickness of covering come within limits where they can be ascribed to incidental differences in material, workmanship, and test conditions, considering that the duration of the test was dependent upon the stability of individual blocks.

(3) CHARACTERISTIC FIRE EFFECTS.—The gypsum coverings failed due to checking, shrinking, and disintegration of the blocks which caused them to loosen and fall off. Characteristic heat effects are shown in Fig. 57. The process responsible for these effects consists mainly in the transformation of hydrated gypsum of the formula, $Ca\ SO_4+2H_2O$, to anhydrous calcium sulphate, by evaporation of the chemically combined water.

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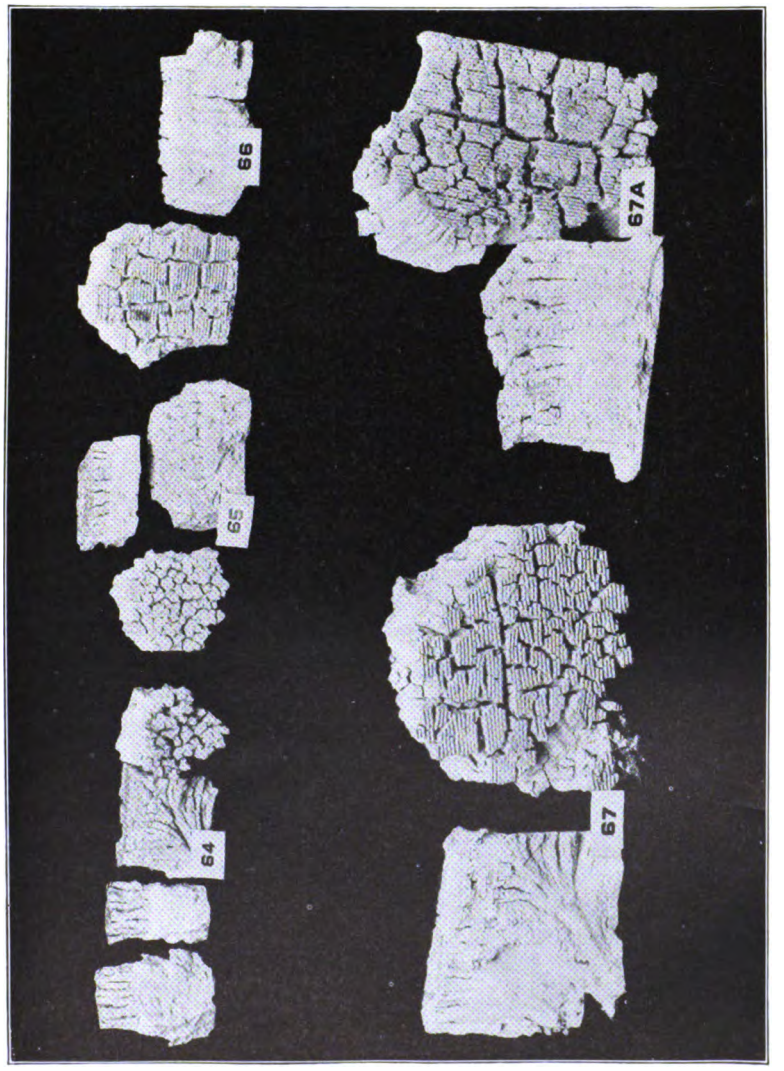

FIG. 57.—*Blocks from gypsum coverings after test*

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Failure of the column occurred within 20 to 40 minutes after the first blocks had fallen. The interval between maximum expansions and failure was relatively short, due to the rapid temperature rise in the steel and the low load-carrying capacity of the covering material that remained in place.

(4) HEAT-INSULATING PROPERTIES.—The maximum temperature in the steel up to the point where the blocks began to fall off was generally below 150° C (302° F), which was much lower than those obtaining in comparable tests with the other covering materials after the same duration of fire exposure (Figs. 127 to 130). The high heat-insulating value of gypsum is due in part to the heat consumed by the change in crystalline structure noted above.

(5) EFFECT OF WATER APPLICATION.—In the case of the 2-inch. protection the effect of the first 2 minutes of the water application was confined to washing away of the partly calcined gypsum near the outer face of the covering, all blocks remaining in place. During the third minute most of the blocks on the sides on which water was applied were carried down along with portions of the poured filling (Fig. 86, p. 245).

On the 4-inch protection the water application of 5-minutes duration washed away the gypsum on three sides to a depth of 1 inch from the surface, all blocks remaining in place.

(i) REINFORCED-CONCRETE COLUMNS

In the reinforced-concrete columns of the fire-test series the coarse concrete aggregates used were Chicago limestone and New York trap rock, and the application of the results should be limited to columns made with these concrete aggregates. Comparisons given in paragraph (*e*) above on the behavior of concrete made with these and other aggregates and applied in coverings for steel columns indicate that less favorable results would be obtained with some of them when applied in reinforced-concrete columns than was obtained with the columns tested. Also in the fire and water tests, the sections of the columns made of siliceous gravel concrete developed much greater disruptive effects during the relatively short fire exposure preceding the water application than the sections made of limestone or trap-rock concrete. The behavior of limestone and trap-rock concrete in tests of reinforced-concrete columns was similar to that of the corresponding concrete of the column coverings, little cracking or spalling of consequence occurring before failure.

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(1) INFLUENCE OF CONCRETE AGGREGATE.—The limestone-concrete columns all withstood the 8-hour fire test and while hot sustained loads exceeding twice the load applied in the 8-hour period. The two vertically reinforced trap-rock concrete columns failed after 7 hours 23 minutes and 7 hours 57 minutes, respectively, and the hooped column withstood the 8-hour fire test and failed under a load about 25 per cent greater than the load sustained during the fire test (p. 113). A 2-inch thickness of concrete next to the surface was assumed as covering in all cases and not included in the area used in computing working loads. The difference in results within the group can be attributed to concrete aggregate, the other incidental factors being comparable or favoring the tests giving the lower results. The trap-rock concrete fused and fluxed at some points to a depth of about 1 inch, which undoubtedly affected the time to failure to some extent. The results obtained with the concrete of both aggregates show a high degree of fire resistance.

(2) EFFECT OF FORM OF COLUMN AND REINFORCEMENT.—No effects due to shape of column or form of reinforcement were evident, differences in results being within the limits of incidental variations in test columns and test conditions.

No line of cleavage outside of the wire reinforcement was found after test in the hooped column of limestone concrete, except in the immediate region of failure, where it was apparently induced by the strains that developed when the column failed. In the case of the corresponding trap-rock concrete column, more evidence indicating separation of the outer protection from the core at the line of the reinforcement was found, effects which may in part have been caused by the fire exposure.

(3) RECOVERY OF STRENGTH AFTER FIRE TEST.—One length of each of the hooped reinforced-concrete columns about 3 feet long was cut outside of the failure region in the fire test and subsequently tested in compression. The limestone-concrete specimen sustained a total load of 517 000 pounds as against 243 000 pounds immediately following the fire test, and the trap-rock concrete specimen 342 000 pounds, compared with 163 000 pounds at the end of the fire test. While a portion of the difference may be due to initial variations in the strength of the concrete, the greater part can be ascribed to recovery in strength of concrete and reinforcement.

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(4) EFFECT OF WATER APPLICATION.—The concrete of the columns subjected to fire and water tests was placed in three sections to permit using two or three kinds in each column.

In the case of the square, vertically reinforced column, No. 111, the water carried away the concrete at the corners outside of the bars and pitted the concrete on the most exposed face to depths of from ⅛ to 1 inch for the limestone concrete and to a depth of 2 inches for the Meramec River gravel concrete in the middle section (Fig. 88, p. 247).

In the round, vertically reinforced column, the limestone concrete was pitted to a depth of 1 inch, and some of the concrete in the upper portion of the Joilet gravel concrete section was carried away. In the middle section, consisting of Meramec River gravel concrete, the outer concrete was stripped off by the water, exposing the reinforcing bars on two sides. In this, as in the preceding test, large cracks had formed in the concrete of the middle section during the fire period.

In the fire and water test of the hooped reinforced-concrete column, the water stripped the Meramec River gravel concrete and the granite concrete from the wire reinforcement on three sides during the first 15 seconds of the water period (Fig. 89). Spalling of concrete had exposed portions of the reinforcement in the middle section during the fire period. Further application of water caused stripping of reinforcement in the upper section of trap-rock concrete and increased the effects in the lower sections.

The condition of none of the reinforced concrete columns was such as to cause apprehension of early failure on being again exposed to fire after the water test, since the proportion of the load normally carried by the steel reinforcement was not large. Load tests to failure made after the water test gave factors of safety of over four as based on the calculated working load.

(*j*) TIMBER COLUMNS

Six tests of timber columns were made, four being tested unprotected. One was protected by a single layer of Portland-cement plaster on metal lath and one by a single layer of gypsum wall board. Two species of wood, longleaf pine and Douglas fir, and two types of post cap details were employed (Fig. 10).

(1) UNPROTECTED TIMBER COLUMNS.—The time to failure of the unprotected timber columns varied from 35 to 50 minutes,

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failure occurring in all cases at the bearings on the steel or cast-iron cap introduced near the top of the column (Figs. 80 to 82). The deformation at the bearing increased rapidly after the first 20 minutes, and at the failure the consequent depression equaled 3 inches or more (Fig. 47, p. 126).

The average time to failure in the tests with longleaf pine was nearly the same as that obtained with Douglas fir, although the tests were hardly comparable on account of the higher moisture content of the Douglas fir in the condition tested (Table 2, p. 30).

The columns with the cast-iron cap and pintle stood up 7 minutes and 15 minutes longer than the columns with the steel-plate cap, the deformations sustained by the wood at the cap before failure ensued being larger in the case of the cast-iron bearing.

(2) PROTECTED TIMBER COLUMNS.—The outer coat of the plaster on parts of the lower half of the metal lath and plaster protection (test No. 78) spalled during the first half hour of the fire test, followed later by local buckling out of lath and supporting channels. Flames from the column issued through this opening before the end of the first hour, and about 15 minutes later flames issued from cracks in the plaster around the column cap. Crushing of the wood at the bearing due to heating of the cap caused a rapid rate of depression, beginning at 1 hour 40 minutes (Fig. 47), failure accompanied by fracture of the cap occurring at 2 hours 15 minutes (Fig. 79, p. 238).

The wall board consisted of gypsum-plaster filling between paper facings. At 20 minutes flames issued from horizontal cracks in the plaster board. The board began to fall off the body of the column at about 40 minutes, and at 54 minutes nearly one-half of the covering had fallen. At 32 minutes part of the covering fell off the flanges of the steel-plate cap, the failure at 1 hour 13 minutes being due to slipping at the cap bearing same as for the unprotected timber columns (Fig. 81, test No. 80).

The protections increased the ultimate resistance of the columns by 100 to 200 per cent as compared with that in the unprotected condition. Protections on timber columns should be applied with due consideration for possible deterioration of the wood that may under certain conditions be induced by the presence of the coverings, particularly when the wood is not fully seasoned or where it is exposed to dampness.

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(3) STRENGTH AFTER FIRE TEST.—The failure being localized at the bearing did not develop the full resistance of the column. The wood was burnt and charred to depths of ¼ to 1¼ inches, involving reductions in effective area of 29 to 55 per cent. One specimen 3 feet long was cut from each of the tested columns and tested in the compression about 10 weeks after the fire test. The results of the tests are given in Table 45, where also are given results of tests on comparable specimens of unburnt timber. The results indicate a reduction in the total load approximately in proportion to the reduction in area, the average of the maximum unit loads sustained by the burnt and unburnt timber of each species being nearly equal.

The strength remaining in the timber at the end of the fire test was considerably lower than the values given in Table 45, due to its heated condition.

TABLE 45.—COMPRESSIVE STRENGTH OF TIMBER AFTER FIRE TEST

Specimens 3 ft. long

Specimen No.	Species	Section Before Test		Section After Test		Reduction, Per cent	*Moisture Content, Per cent	Maximum Load	
		Outside Dimensions, In.	Area, Sq. In.	Outside Dimensions, In.	Effective Area, Sq. In.			Total, Lb.	Lb. per Sq. In.
78-1	Longleaf pine.	11.3 by 11.5	129	7.9 by 8.4	58	55	15.5	370,000	6380
79-1	Longleaf pine.	11.2 by 11.2	125	8.8 by 9.1	76	44	14.4	381,000	5010
80-1	Longleaf pine.	11.2 by 11.3	126	9.0 by 9.0	75	40	15.8	399,000	5320
81-1	Longleaf pine.	11.4 by 11.4	129	9.0 by 9.1	77	40	16.2	475,000	6170
	Longleaf pine.	Average...	.....	.....	.....	.....	15.5	.....	5720
82-1	Douglas fir....	11.4 by 11.4	129	9.5 by 9.5	87	33	22.7	324,000	3725
83-1	Douglas fir....	11.4 by 11.4	129	10.6 by 10.7	91	29	18.8	400,000	4395
	Douglas fir....	Average...	.....	.....	.....	.....	20.7	.....	4060
P-1	Longleaf pine.	11.2 by 11.4	127	Specimen of	unburnt	timber.	19.3	750,000	5765
F-1	Douglas fir....	11.4 by 11.4	129	Specimen of	unburnt	timber.	18.9	502,000	3890
F-2	Douglas fir....	11.4 by 11.4	129	Specimen of	unburnt	timber.	18.9	500,000	3875
	Douglas fir....	Average...	.....	.....	.....	.....	18.9	.....	3880

*Determined from samples taken near point of failure, dried to constant weight at 100° C.

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XIII. FIRE-RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS

The results of the tests in point of time to failure will be summarized in terms of hours and minutes of fire resistance afforded by the different types of columns and protections tested.

1. BASIS OF DERIVATION

(a) METHOD OF COMPUTATION

A given resistance period is taken to hold, if the time to failure in the fire test or the average of the time to failure in a group of similar tests is equal to 1½ times the given resistance period. The deduction of one-third of the test duration is made to allow for incidental variations in material and workmanship of columns and coverings and differences in load and fire conditions that cause variations in results with nominally comparable columns. Individual test results within a given group may be below this limit, but not below the designated resistance period.

(b) INTERVALS

Resistance periods are taken 5 minutes apart up to one-half hour, at 15-minute intervals from one-half hour to 1 hour, at one-half hour intervals from 1 to 4 hours, and at 1-hour intervals from 4 to 8 hours. A variation of one-fourth of an interval is allowed, a given even value of the resistance period being taken if the period, computed according to the method given above, is not more than one-fourth of the interval below the given period value.

(c) TABLE OF FIRE-RESISTANCE PERIODS

A tabulation of the fire-resistance periods obtained from the results of the present series of tests is given in Table 46.

(d) DERIVATION OF METHOD

The method of computation is obtained from comparison of duplicate or nearly duplicate column tests considered in connection with conditions affecting comparable column constructions in buildings. In the fire tests, the maximum range in time to failure of unprotected structural steel columns was 49 per cent, comparable tests of partly protected steel columns gave a range in values of 24 per cent, plaster on metal lath protections, 15 per cent,

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TABLE 46.—FIRE RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS

Type of Column	PROTECTION		Minimum Area of Solid Material, Sq. In.	Nominal Thickness of Protection, In.	Fire Resistance Period
	Material	Details			
Structural steel	Unprotected	Minimum metal thickness, .20 in.	8	.....	10 min.
Structural steel, solid section	Partly protected by filling reentrant spaces with concrete. Concrete aggregates; limestone, calcareous gravel, trap rock, granite, sandstone, or hard coal cinder	Mixture, 1:6 or 1:8. Concrete tied with vertical and horizontal steel ties	35	.....	½ hr.
Structural steel, solid section	Partly protected by filling reentrant spaces with concrete. Aggregates; limestone, calcareous gravel, or trap rock	do.....	60	.....	¾ hr.
Structural steel, open latticed section	Partly protected by filling interior and reentrant spaces with concrete; trap rock aggregate	Mixture, 1:6. Filling extends to outside rivets and covers lattice and main members	120	.....	2 hr.
Structural steel, open latticed section	Partly protected by filling interior and reentrant spaces with concrete; limestone or calcareous gravel aggregate	do.....	120	.....	2½ hr.
Structural steel	Portland cement plaster on metal lath	Proportion of plaster, 1:½:2½, Portland cement, hydrated lime, and sand	40	1 layer 1 in.	¾ hr.
Structural steel	do.....	do.....	80	2 layers each ⅝ in.	1½ hr.
Structural steel	Concrete; siliceous gravel aggrogate	Mixture, 1:6. Concrete tied with steel ties or wire mesh, equivalent to not less than No. 5 (B. & S. gage) wire on 8-in. pitch	100	2	1 hr.
Structural steel	do.....	do.....	200	4	2½ hr.
Structural steel	Concrete; granite, sandstone, or hard coal cinder aggregate	Mixture, 1:6. Concrete tied as above	100	2	2½ hr.
Structural steel	do.....	do.....	140	3	3½ hr.
Structural steel	do.....	do.....	200	4	5 hr.
Structural steel	Concrete; trap rock aggregate	Mixture, 1:6. Concrete tied as above	100	2	3 hr.
Structural steel	do.....	do.....	140	3	4 hr.
Structural steel	do.....	do.....	200	4	5 hr.
Structural steel	Concrete; limestone or calcareous gravel aggregate	Mixture, 1:6. Concrete tied as above	100	2	4 hr.
Structural steel	do.....	do.....	140	3	6 hr.
Structural steel	do.....	do.....	200	4	8 hr.

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TABLE 46.—FIRE RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS—Continued

Type of Column	PROTECTION		Minimum Area of Solid Material, Sq. In.	Nominal Thickness of Protection, In.	Fire Resistance Period
	Material	Details			
Structural steel, solid section	Hollow tile; semi-fire clay, medium hard. No filling	Mortar joint between tile and column flanges. Outside wire ties	80	2, 3, or 4	1 hr.
Structural steel, solid section	Hollow tile; surface clay or shale. Concrete filling on web sides	do	100	2, 3, or 4	1 hr.
Structural steel, solid section.	Hollow tile, extra heavy; surface clay. Concrete filling on web sides	do	160	3 or 4	1½ hr
Structural steel	Hollow tile; semi-fire clay or surface clay. Concrete filling all around	Outside wire ties	160	2, 3, or 4	2 hr.
Structural steel	do	Metal ties in horizontal joints	160	3 or 4	2½ hr.
Structural steel, solid section	Hollow tile; surface clay. Hollow tile filling	Mortar joint between tile and column flanges and webs. Metal ties in horizontal joints	240	2 layers each 2 in.	3 hr.
Structural steel, solid section	do	Mortar joint between tile and column flanges and webs. Outside wire ties	240	2 layers each 2 in.	1 hr.
Structural steel	Common brick; surface clay	Brick set on edge and end	140	2¼	1 hr.
Structural steel	do	Brick laid on side	240	3¾	5 hr.
Structural steel	Solid gypsum block. Gypsum block or poured gypsum filling	Metal ties in horizontal joints. Mortar joint between blocks and column flanges	130	2	1½ hr.
Structural steel	do	do	180	3	2½ hr.
Structural steel	do	do	240	4	3½ hr.
Round cast iron	Unprotected; unfilled	Minimum thickness of metal, .60 in.	12		20 min.
Round cast iron	Unprotected; interior filled with concrete	do	35		½ hr.
Round cast iron	Portland cement plaster on high ribbed metal lath	Proportion of plaster, 1:$\frac{1}{10}$:2½, Portland cement, hydrated lime and sand	60	1 layer, 1½ in. ½ in. air space	2 hr.
Round cast iron	Concrete; trap rock, granite, or hard coal cinder aggregate	Mixture, 1:7. Concrete tied with steel ties equivalent to not less than No. 5 (B. & S. gage) wire on 8-in. pitch	70	2	2 hr.
Round cast iron	Hollow tile; porous semi-fire clay	Outside wire ties. Mortar joint between tile and column	70	2	2 hr.

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TABLE 46.—FIRE RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS—Concluded

Type of Column	PROTECTION		Minimum Area of Solid Material, Sq. In.	Nominal Thickness of Protection, In.	Fire Resistance Period
	Material	Details			
Steel pipe	Unprotected. Filled with concrete	Concrete mixture, 1:1½:3	35	.....	25 min.
Reinforced steel pipe	Unprotected. Filled with concrete and reinforced in the fill with structural shapes	Concrete mixture, 1:1½:3	45	.....	¾ hr.
Reinforced concrete	Limestone or calcareous gravel concrete	Mixture 1:2:4. Concrete reinforced with vertical bars and lateral ties or hooping	220	2	8 hr.
Reinforced concrete	Trap rock concrete	do.....	220	2	5 hr.
Timber, longleaf pine or Douglas fir	Unprotected	Unprotected cast iron or steel cap	120	.....	25 min.
Timber, longleaf pine or Douglas fir	Gypsum wall board	Cast iron or steel cap	140	1 layer, ⅝ in.	¾ hr.
Timber, longleaf pine or Douglas fir	Portland cement plaster on metal lath	Cast iron or steel cap. Proportion of plaster, 1:½:2½, Portland cement, hydrated lime, and sand	160	1 layer, 1 in. with ¾ in. air space	1½ hr.

and 2-inch concrete protections 25 per cent, all percentages being based on the highest value within the group. The time to failure in three tests of unprotected cast-iron columns differed by less than 1 per cent. In hollow clay-tile protections, excluding Nos. 50 and 50A on account of not being tied, the maximum range in test duration within a group of comparable tests was 30 per cent, and in gypsum-block protections 26 per cent. Unprotected timber columns with exposed steel or cast-iron caps gave a range in time to failure of 30 per cent, part of which was due to difference in bearing details.

In the case of unprotected structural steel columns, the range in shape of section tested was large and a deduction of one-third from the average time to failure gives a period below which test results, obtained under comparable load and fire conditions, are not likely to fall, with the limitations in thickness of metal and area of section given in Table 46. All of the other variations noted above come within 30 per cent of the highest value in the respec-

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tive groups, although it is appreciated that the test duplications were not sufficient in number to develop the full possibility of difference in results; also, that columns and protections in buildings are subject to greater variability than the test columns. The basis of derivation adopted assumes possibility of variation of about 33 per cent above and below an average value, or a total of about 66 per cent of the average. Irrespective of whether or not this range in results would fully develop in a sufficiently large number of comparable cases in buildings, the use of the one-third reduction to obtain minimum ultimate fire-resistance periods is considered justified because of the few number of tests on which most of the resistance periods are based, several of them being derived from single tests, the results of which it is necessary to assume fall within the higher rather than the average or lower range of possible variation.

(*e*) RESISTANCE TO WATER APPLICATION

With reference to fire and water exposure the results are regarded as satisfactory if the applied load was safely sustained during the fire and water periods, and in the case of the protected columns, if the covering remained in place, after a 2-minute period of water application, to such extent as to prevent early failure of the column on being again exposed to fire, precluding in all cases complete removal of the covering from any section of the column.

(*f*) SIZE LIMITATIONS

The derived fire-resistance periods apply most nearly to columns of approximately the same size as those tested and should be applied with considerable caution in connection with smaller columns. Minimum areas with relation to the derived periods are given in Table 46. The studies made on effect of size indicate that the test results can be applied with safety to columns of larger size than those tested.

(*g*) APPLICATION TO BUILDING CONDITIONS

It is believed that the types of columns and protections tested can be applied with confidence in building construction as being able to resist fires of duration equal to the pertaining resistance periods herewith derived for the respective types, provided that reasonable care is taken to identify the material used and to secure a fair grade of workmanship.

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In constructing the coverings and columns, a consistent effort was made to introduce methods and conditions similar to those incident with building construction, and it is thought that the columns as tested are fairly representative of the average attained in good building practice.

The columns tested covered a wide range in material and shape of section, and each class of covering material was represented by its main varieties in common use. In applying the results broadly some difficulty may be experienced due to difference in materials of the same kind or name as occurring in different localities, relative to which some explanatory notes and cautions are given in deriving fire-resistance periods with the respective materials. It is believed that if the materials are properly identified and classed with the corresponding materials employed in the column tests, the resulting difference in fire-resisting properties will not be large, provided the mineral constituents and impurities are within the limits hereafter given.

In the column tests the temperature indicated by the pyrometers, while corresponding on the average quite closely with those on the reference curve (compare reference curve and average curve, Fig. 39), was lower than that of the furnace gases surrounding them, due mainly to radiant heat interchange between the pyrometer and the colder furnace inclosure. The difference was determined to be as large as 150° C for points near the beginning of the test, with gradual decrease to less than 50° C at the end of an 8-hour run (Sec. IX, par. *2d*, Fig. 41). The high intensity of the furnace exposure to which the columns were subjected can be taken as compensating to some extent for differences in material incident with broad application of the test results, and give assurance that such application is justified where the variance from the materials tested is not too large.

The loads applied to the columns during test come within the higher range of normal working loads, and it is believed that the loads imposed on columns under fire conditions in buildings will seldom be much larger, either as caused by floor loads or by unequal expansion of adjacent columns during the fire.

The column coverings had a full and firm bearing at the base of the column and were full and continuous at the top, and therefore took portions of the applied load proportionate to their

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relative area and rigidity with respect to the structural section. Coverings as applied in buildings will sometimes have less firm bearing at the base and be less continuous at the top, due to obstructions to proper placement offered by the floor members. This difference in ability to carry load will affect chiefly the concrete protections and fillings, since the other types have little load-carrying capacity near failure. With concrete protections it will affect only the period between maximum expansion and failure, the metal section assuming almost all of the load during the preceding period, due to its higher rate of expansion. Concrete will develop bond within a relatively short distance with a number of forms of metal section, making the covering in effect a part of the column in the middle portion of its length, even if it is not fully continuous at the ends. It is therefore apparent that the difference between the test condition and the condition of possible occurrence in buildings is not large as far as it concerns the fire resistance of the column, and considering the relatively smaller variations due to other causes generally incident with concrete protections, it can be taken as sufficiently allowed for in the reduction of the time to failure adopted for deriving fire-resistance periods.

The fire resistance afforded by columns is based on the time to failure rather than the useful limit because the former is definitely determined by the test procedure. It is deemed, however, that with the interpretation of test results given above the protection given by the columns and coverings will generally be sufficient to prevent permanent damage of such extent as to require repair or replacement after fire exposures corresponding in duration to the resistance periods here established. Only where very severe local fire exposure is coincident with other unfavorable conditions, such as inferior material and workmanship, will the resistance of the column beyond its useful limit be likely to be developed.

2. DERIVATION OF FIRE-RESISTANCE PERIODS

In deriving the resistance periods the method described in paragraph 1*a* of this section is followed, giving consideration also to effects of water application as explained in paragraph 1*e*. Interpolations for covering thicknesses intermediate between those tested are made where justified by the test results. Limited extension of results to materials not included in the particular

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combination considered, but giving similar test characteristics in other related tests of this series, is made in the case of concrete and hollow clay-tile protections (Table 46, p. 197–199).

For columns with square or rectangular protections, all portions of the main members are assumed covered by not less than the nominal thickness of material specified. With round coverings, on columns of square or rectangular outline, the distance from the surface to the edge of the main column members may be reduced somewhat, provided the resulting cross-sectional area is not less than that of a square or rectangular protection of the same nominal thickness.

Details having relatively large areas, such as lattice bars and splice plates, need to be covered by the nominal thickness specified. For smaller areas as of rivets and extreme edges of bracket or supporting angles, the covering thickness may be reduced to 1 inch.

Where partitions or pipe inclosures are supported by column coverings the connection should be made so that the stability of the covering is not affected by buckling or failure of the supported walls. Pipes should not be placed within the thickness of covering required for the given resistance periods nor incorporated in the covering in such manner as to dislodge any essential portion thereof by expansion or buckling incident with normal use or with fire conditions.

(*a*) UNPROTECTED STRUCTURAL-STEEL COLUMNS

As based on test Nos. 1 to 8, and with not less than the minimum metal thickness and area of section given in Table 46, unprotected structural-steel columns give *10-minute* fire resistance.

The low degree of resistance afforded indicates unsuitability for use where fires of any appreciable degree of intensity and duration are possible.

(*b*) PARTLY PROTECTED STRUCTURAL-STEEL COLUMNS

The protection consists of concrete that fills the interior and reentrant portions to the outside of the extreme metal on the main members. Except on the open sides of latticed columns, the concrete is secured with horizontal and vertical metal ties.

(1) SOLID-SECTION COLUMNS.—Structural-steel columns of solid rolled or riveted section, whose reentrant portions are filled with

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concrete made with any of the aggregates included in the present series of tests, except highly siliceous sand and gravel, mixed in proportion of 1 part Portland cement to not more than 8 parts fine and coarse aggregates combined, and with combined area of steel and concrete not less than 35 square inches, give *one-half hour* fire resistance. Similar columns filled in the same manner with limestone, calcareous gravel, or trap-rock concrete, and with area of filled column not less than 60 square inches, give *three-fourths hour* fire resistance.

The one-half hour period is based on test Nos. 15, 16, and 17, and the three-fourths hour period on test Nos. 14, 20, and 21.

(2) OPEN LATTICED SECTION.—Structural-steel columns of open latticed section, with the interior and reentrant portions filled with concrete of proportion 1 part Portland cement to not more than 6 parts fine and coarse aggregates combined, and with total area of not less than 120 square inches, give *2-hour* fire resistance if concrete is made with trap-rock aggregate and *3½-hour* fire resistance if limestone aggregate is used.

The above conclusions are based on test Nos. 18 and 22, the higher resistance over the solid section columns being due to absence of large areas of exposed metal. It is essential to use concrete that is not subject to cracking or spalling, since the covering on parts of the main members is usually little more than 1 inch thick and not tied.

(c) STRUCTURAL-STEEL COLUMNS WITH PLASTER ON METAL-LATH PROTECTIONS

The plaster can be applied on either expanded metal or woven-wire lath. The layer thickness is measured from the inner line of the lath. The proportion of the plaster considered is 1 part Portland cement to not more than 2½ parts medium-coarse sand, with lime added equal to one-tenth of the cement volume. The periods derived are based on test Nos. 23 to 26 and 110, in which the above proportion was used. The effect on fire-resistive properties produced by varying the lime content is otherwise not known.

(1) SINGLE-LAYER PROTECTION.—Structural-steel columns protected by a layer of Portland-cement plaster 1 inch thick applied on metal lath give *three-fourths hour* fire resistance.

(2) DOUBLE-LAYER PROTECTION.—Structural-steel columns protected by two layers of Portland-cement plaster each ⅞ inch thick with ¾-inch air space between layers give *1½-hour* fire resistance.

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(d) CONCRETE PROTECTIONS ON STRUCTURAL-STEEL COLUMNS

The protections, preferably placed without construction joints, should extend through floors, where the floor filling is of other material than concrete. Lateral ties consisting of iron or steel wires or mesh, embedded in the covering and extending to within about 1 inch from its surface, are to be equivalent to not less than a No. 5 (B. and S. gage) steel wire wound spirally on a pitch of 8 inches.

The proportion of mixture taken to hold is 1 volume part Portland cement to 6 parts fine and coarse aggregates combined, 94 pounds of the cement being taken as its weight per cubic foot. The effect of reducing the cement content of the mixture was not definitely determined in the tests, although some reduction in fire resistance was noted as between the 1:6 and the 1:8 proportion, estimated as equivalent to reductions in the resistance periods of not more than ½ hour for periods up to 3 hours and possibly as much as 1 hour for the longer resistance periods.

Limitations relative to size are given in Table 46 (p. 197). For columns of dimensions much larger than the given minimum, the periods apply with a considerable margin of safety.

(1) SILICEOUS-GRAVEL CONCRETE PROTECTION.—Structural-steel columns covered with a 2-inch thickness of concrete made with gravel aggregate of high silica content give *one-hour* fire resistance, and with similar 4-inch protections give *2½-hour* fire resistance.

Since the failure of coverings of this type of concrete is dependent on stability of portions thereof, no adequate basis is present for estimating the value of the resistance period for thicknesses intermediate between 2 and 4 inches.

The periods are based on test Nos. 39 and 45, where the aggregate consisted mainly of chert grains and pebbles. Since this form of silica develops as great or greater disruptive effects on exposure to fire as quartz, flint, and related minerals, the derived protection periods apply generally where concrete made with all common forms of siliceous aggregates is used.

(2) GRANITE, SANDSTONE, OR HARD-COAL CINDER CONCRETE PROTECTION.—Structural-steel columns covered with 2-inch protections of granite, sandstone, or hard-coal cinder concrete give *2½-hour* fire resistance, with 3-inch coverings give *3½-hour* fire resistance, and with 4-inch coverings give *5-hour* fire resistance.

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Tests were made of 2-inch coverings of concrete made with sandstone and cinder (Nos. 31, 32, 32A, 43, 44, and 104), and in the 4-inch thickness, with granite as the coarse aggregate (test Nos. 34, 34A, and 103). The extension of the results with sandstone and cinder concrete to cover thicknesses heavier than 2 inches, and of granite concrete to cover thicknesses lower than 4 inches, is based on relative effectiveness with respect to trap-rock concrete in coverings (test Nos. 29, 36, and 37) of comparable thickness.

The granite used in the concrete of the column coverings tested contained 35 per cent quartz and about 60 per cent feldspars, chiefly orthoclase (Table 20, p. 345), the size of crystals ranging from 0.03 inch to 0.35 inch. The range in quartz content of granites is generally from 20 to 40 per cent, with extremes from 5 to 50 per cent. In crystal size the general range is from 0.02 inch to 0.40 inch, with extremes up to 2½ inches. It appears, therefore, that the granite tested came within the higher range in quartz content and had crystals of average size. To what extent changes in mineral composition and crystal size of granite affect the fire-resistive properties of concrete made with it is not known, although it would be expected that those of lower quartz content will be the least subject to destructive fire effects.

The Berea sandstone used in these tests was a silica cemented stone of average hardness consisting almost wholly of subangular grains of quartz. Some disintegration from heat due to loss of cementing properties was noted, but not enough to affect the fire-resisting properties, the principal fire effects being cracking and dislodgment of relatively large masses, induced presumably by the high quartz content.

Among sandstones used as concrete aggregate, quartzite has all pore spaces filled with silica cement, resulting in a hard, homogeneous, quartz mass. Flame tests made on small specimens of the stone do not indicate any greater disruptive effects than those incident with similar tests of other sandstones.

Silica cemented sandstones range in hardness from compacted sand to a condition approaching quartzite, all being of high silica content, usually over 95 per cent. The softer grades are generally eliminated from use as concrete aggregate on the score of deficient strength and hardness.

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The lime-cemented sandstones range in mineral composition from nearly pure sandstone to sandy limestones carrying only small amounts of silica. The iron (limonite) and clay cemented sandstones, of which the brownstones are representative, carry variable amounts of feldspar and mica, the range in free quartz content being from 50 to 90 per cent. The bluestones of New York, Pennsylvania, and West Virginia contain from 20 to 60 per cent quartz, the other minerals being mainly feldspar and hornblende.

While not possible to predict fully the behavior of combinations of minerals on exposure to fire, it is probable that the addition of calcareous, clayey, or feldspathic minerals to quartz to form the sandstones noted above will decrease rather than increase the cracking incident with fire exposure of purer forms of the rock.

The hard-coal cinder contained about 10 per cent unburned coal and 5 per cent ash, and to obtain comparable results these percentages should not be greatly exceeded.

(3) TRAP-ROCK CONCRETE PROTECTION.—Structural-steel columns protected by 2-inch coverings of trap-rock concrete give *3-hour* fire resistance, with 3-inch coverings give *4-hour* fire resistance, and with 4-inch coverings give *5-hour* fire resistance (test Nos. 29, 36, 37, 40, 101, 103, and 104).

The periods for the 2 and 4 inch covering thicknesses were derived directly from the test results, and that for the 3-inch thickness taken as their average.

Trap rock is a dark-colored, fine-grained igneous rock that does not vary greatly in mineral composition and carries at most only a trace of quartz (Table 20). The term does not include the hard sandstones known under the same name in some localities.

(4) LIMESTONE OR CALCAREOUS-GRAVEL CONCRETE PROTECTION.—Structural-steel columns protected by 2-inch coverings of limestone or calcareous-gravel concrete give *4-hour* fire resistance, with 3-inch coverings give *6-hour* fire resistance, and with 4-inch coverings give *8-hour* fire resistance.

Tests were made of 2-inch coverings of limestone and calcareous-gravel concrete (test Nos. 28, 28A, 30, 38, 101, 102, and 104) and of 4-inch coverings of 1:6 and 1:8 limestone concrete (test Nos. 33, 33A, 35, 41, 42, and 103). The period for the 2-inch thickness was derived directly from the test results. The columns with the

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4-inch covering all withstood the 8-hour fire test, and while hot sustained such large additional loads as to justify the conclusion that in the lower range of results with comparably protected columns the working load will be sustained during an 8-hour fire period.

The limestone and the calcareous gravel of the tests were dolomitic and contained about 5 per cent of clayey impurities (Table 20). It is thought that the results apply without modification where high calcium limestone or gravel of nearly the same purity is used. The silica impurities in limestone occur as free silica (quartz, chert, flint, and opal) and silica combined in clay and other silicates. Most limestones carry from 5 to 10 per cent of combined clayey impurities, but not many have more than 10 per cent of free silica, and within the given limits these impurities are not deemed objectionable for stone or clean gravel used as concrete aggregate in fire-resistive construction. Some limestones contain higher percentages of free silica and particularly the waste from limekiln and cement plant quarries is likely to be high in chert.

Gravels in the glaciated area, which includes nearly all of the territory north of the Missouri and Ohio Rivers, New England, New York, northern Pennsylvania, and northern New Jersey, may contain material from any point to the northward and formed under the most diverse conditions will vary greatly in mineral composition both vertically and horizontally. This also applies to the gravels of the Great Lakes and the ocean gravels as far south as New York. If the superior fire-resistive qualities of concrete made with calcareous gravel is to be assured, or the highly siliceous gravel avoided, it is necessary to identify the material in each bank by means of suitable chemical or mineralogical analyses at successive times as the development proceeds.

Outside of the glaciated area, the river and shore gravels are more uniform, although varying with changes in the rock formations of the drainage area or headlands.

The calcareous content of the sands (Fox River and Joliet) used in combination with the limestone and calcareous gravel was higher than the combined quartz and chert content (Table 20). While all sands are likely to contain considerable silica, and the extent of their influence on the thermal properties of concrete made with them is not known, it is thought best to avoid using highly siliceous

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sands in combination with limestone and calcareous gravel where the full fire-resistive value of the concrete is to be utilized. As used with the other coarse concrete aggregates, the mineral composition of the sand is of minor importance, provided it meets the common requirements for a good concrete sand. The sand in a given deposit is generally more uniform in composition than the larger-sized material.

(e) HOLLOW CLAY-TILE PROTECTIONS ON STRUCTURAL-STEEL COLUMNS

The tile is assumed applied in courses about 12 inches high with fairly full horizontal and vertical mortar joints between the tile units. The mortar should also fill the space between the tile and column flanges except where it is filled with concrete. The proportion of mortar used in the tests was 1 volume part Portland cement to 1 of stiff slaked lime, and 4 of fine sand. While the influence of mortar proportion on the effectiveness of the covering is not definitely known, the test results are not fully applicable unless mortar having about the same content of cementing materials as that given above is used, and particularly the Portland cement should not be reduced below the proportion given. As an aid in assigning fire-resistance periods for combinations of tile and details of application not included in the tests, it can be stated that in the tests the semifire-clay tile burnt to medium hardness displayed the best fire-resistance properties, followed in order by tile of hard burnt semifire clay, of surface clay, and of shale.

The coverings tested were generally tied with one No. 12 (B. & S. gage) iron or steel wire placed tightly around the outside of each course, or by strips of woven wire of ⅜-inch mesh, placed in the horizontal joints and lapping at the corners. The latter method is considered superior to the outside wire ties and can be substituted for them where desired. The difference in effectiveness of the two methods is not considered so large as to justify requiring the mesh except for tile subject to severe disruptive effects on exposure to fire and where the higher resistance periods are to be attained. Other forms of interior ties may be used if equivalent in effect to the wire mesh.

In assigning fire-resistance periods to given protections, consideration is given to the effect of area of solid material in the cross section of the covered column, having reference to the minimum values given in Table 46 (p. 198).

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The proportion of mixture for the concrete filling was 1 part Portland cement to not more than 8 parts fine and coarse aggregates combined, the filling being placed after the tile was set. Any aggregate used in these tests may be employed except highly siliceous gravel.

(1) UNFILLED PROTECTION.—Structural-steel columns with hollow clay-tile protections of 2, 3, or 4 inch semifire-clay tile of medium hardness, tied with outside wire ties, give *one-hour* fire resistance.

This period is based on results of fire tests Nos. 48 and 49 taken together with fire and water tests Nos. 105 and 107. The tile in these tests developed fewer disruptive effects on exposure to fire than any other kind tested. Tile more sensitive to abrupt temperature change is not adapted for use in unfilled protections, as the cracking and spalling resulting after a short fire exposure allows the heat to readily reach the steel, also, on application of water, parts of the impaired covering are liable to be carried away, leaving the metal unprotected against a possible recurring fire.

The use of unfilled protections should generally be avoided as the filling materially increases their stability and insulating value.

(2) SHALE OR SURFACE-CLAY TILE PROTECTION WITH CONCRETE FILLING REENTRANT SPACES.—Structural-steel columns with hollow clay-tile protections of 2, 3, or 4 inch shale or surface-clay tile, tied with outside wires and with concrete filling the reentrant spaces between tile and column web, give *one-hour* fire resistance (test Nos. 50, 50A, 52, 53, and 106).

Similar protections of 3 or 4 inch surface-clay tile with heavy shells and webs give *1½-hour* fire resistance (test Nos. 51 and 51A).

(3) SEMIFIRE-CLAY OR SURFACE-CLAY TILE PROTECTION WITH FULL CONCRETE FILLING.—Structural-steel columns with hollow-tile protections of the given types of clay, set 1 inch away from the flanges and edges, with concrete filling all spaces between the steel and the tile, give *2-hour* fire resistance where outside wire ties are used (test Nos. 54, 55, and 57), and with wire mesh in the horizontal joints give *2½-hour* fire resistance (test Nos. 56 and 77).

The thickness of tile, using outside wire ties, may be either 2, 3, or 4 inches. With the wire mesh the thickness should preferably be not less than 3 inches to allow ties of sufficient width to be used.

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(4) DOUBLE 2-INCH TILE PROTECTION.—Structural-steel columns with protections of two layers of 2-inch surface-clay tile, and with the space between the inner tile layer and the column filled with tile set in place, give *3-hour* fire resistance when the covering is bonded with wire mesh in the horizontal joints and *1-hour* fire resistance if tied with outside wire ties.

The periods are based on test Nos. 58 and 59, which indicated a marked difference in results due to the method of tying the surface-clay tile applied in the given type of protection. Concrete filling may be substituted for the tile filling if desired.

(*f*) BRICK PROTECTIONS

The fire resistance periods are derived from the results of test Nos. 68 and 69. The same qualifications relative to mortar apply as for the hollow clay-tile protections, as also the size limitations given in Table 46 (p. 198).

Structural-steel columns thus protected with common surface-clay brick, set on edge and end outside of the flanges and edges and filling the whole space to the steel, give *1-hour* fire resistance, and with brick laid flat on side to form a solid protection of about 4-inch thickness *5-hour* fire resistance is developed.

(*g*) GYPSUM-BLOCK PROTECTIONS

The derived resistance periods apply to columns covered with solid gypsum block set in mortar of proportion, 1 part calcined gypsum to 3 parts fine sand, the blocks being bonded with strips of corrugated iron or wire mesh placed in the horizontal joints over all vertical joints. The space between the outer blocks and column can be filled with gypsum blocks set in place or with poured filling consisting of calcined gypsum, broken gypsum blocks, and sand, mixed with enough water to enable proper placement.

Structural-steel columns with protections of 2-inch solid gypsum blocks placed according to the above details give *1½-hour* fire resistance (test Nos. 65, 66, and 108), and with similar protections of 4-inch solid gypsum blocks give *3½-hour* fire resistance (test Nos. 64, 67, 67A, and 109). Interpolation between the two thicknesses appears justified, the protection made with the 3-inch solid block giving accordingly a resistance period of *2½ hours*.

The area limitations given in Table 46 apply less rigidly to gypsum-block protection than to most other types, because the

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failure is induced by loss of stability of the blocks after a given fire exposure and not to normal transmission of heat through the covering.

(A) CAST-IRON COLUMNS

(1) UNPROTECTED COLUMNS.—With the minimum area and wall thickness given in Table 46, unprotected cast-iron columns give *20-minute* fire resistance.

The fire tests (Nos. 9. 10, and 10A) were made on round columns of about ¾-inch wall thickness, and gave uniform results, although they were not sufficient in number to develop possible variations due to quality of metal and differences in wall thickness. The fire and water tests, while inducing large permanent deflections, did not cause the columns to lose ability to sustain working loads.

Filling the interior with concrete (test No. 11) gives an increase in resistance of 10 minutes or a total period of *one-half hour*.

(2) PLASTER ON METAL-LATH PROTECTION.—A single heavy layer of Portland-cement plaster of 1½-inch average thickness applied on high-ribbed metal lath to round cast-iron columns gives a fire resistance of *2 hours* (test No. 27).

The same proportion of plaster applies as given above under (*c*) for protections on steel columns. To attain the required average layer thickness, the surface of the covering will extend about 2 inches outside of the metal on account of voids inside of the lath.

(3) CONCRETE PROTECTION.—Round, cast-iron columns with 2-inch protections of trap-rock, granite, or hard-coal cinder concrete, give *2-hour* fire resistance.

The period is based on test No. 47 where cinder concrete was used. Extension to include concrete made with the other aggregates is made because of characteristics similar to cinder concrete developed by them in other tests of concrete protection. Limestone and calcareous gravel concrete would on the same basis give longer resistance periods. The proportion of the concrete can be taken as 1 part Portland cement to not over 7 parts aggregate. Relatively small-sized aggregates need to be used, to enable good placement of the covering, and with some aggregates it may be necessary to increase the thickness of the covering to secure this result. The ties, equivalent to not less than No. 5 (B. & S. gage) steel wire on 8-inch vertical pitch, are to be supported about 1 inch away from the column.

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(4) HOLLOW CLAY-TILE PROTECTION.—Cast-iron columns covered with 2-inch curved porous semifire-clay tile, with ¾-inch mortar thickness between tile and column, and tied with outside wire ties, give *2-hour* fire resistance (test Nos. 62 and 63).

The same details of application can be taken to hold as for hollow-tile protections on structural-steel columns.

(*i*) UNPROTECTED PIPE COLUMNS

Unprotected columns consisting of steel or wrought-iron pipes not smaller than the standard 6-inch pipe size, with filling of concrete, give *25-minute* fire resistance.

Pipe columns not smaller than the 7-inch standard pipe size, filled with concrete and reinforced in the fill with structural shapes, give *three-fourths hour* fire resistance.

The proportion of the concrete filling is taken to be 1 part Portland cement to not more than 4½ parts fine and coarse aggregates combined (Table 46, p. 199).

The conclusions relative to pipe columns are based on test Nos. 12 and 13, made, respectively, on a 7-inch plain pipe column and on an 8-inch reinforced pipe column, both being standard new steel pipes with wall thickness of about 5/16 of an inch. Extension of the result with the former to make it applicable to the 6-inch pipe size and of the latter to apply to the 7-inch size, appears justified.

(*j*) REINFORCED-CONCRETE COLUMNS

The proportion of the concrete for which the derived periods hold is 1 part Portland cement to 6 parts fine and coarse aggregates combined, the coarse aggregates being trap rock or limestone (cf. Sec. XII, par. 3*i*). The same considerations relative to possible variations in the aggregate hold as for concrete protection.

The columns may be round or square and reinforced with vertical bars held by lateral ties spaced not farther apart than 12 inches or by vertical bars and spirally wound wire hooping, all reinforcements to be covered by not less than 2 inches of concrete placed integrally with the structural portion of the column.

Under these conditions reinforced-concrete columns made with limestone or highly calcareous gravel-aggregate can be taken as giving *8-hour* fire resistance (test Nos. 70, 72, 74, 111, and 112), and made with trap-rock aggregate, *5-hour* fire resistance (test Nos. 71, 73, 75, and 113) (Table 46, p. 199).

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The limestone-concrete columns withstood the 8-hour fire test, and while hot sustained ultimate loads of from a little less than twice to nearly three times the load sustained during the 8-hour period. This gives reasonable assurance that all similarly constructed columns, using a properly identified limestone aggregate, will sustain working load until the end of an 8-hour fire period, taking into account all probable variations. Calcareous gravel can be used in place of limestone if the free silica and other impurities are within the limits given in paragraph (*d*4) (p. 208).

Two of the trap-rock concrete columns failed during the 8-hour period and the other withstood the 8-hour fire test and about 25 per cent additional load at the end of this period. The fire-resistance period is therefore almost fully determinate on the basis of test duration.

(*k*) TIMBER COLUMNS

With unprotected or protected timber columns having exposed cast iron or steel caps, the resistance to fire is limited by failure at the bearings before the full resistance of the timber has been developed elsewhere. (Cf. Sec. XII, pp. 167 and 194). The resistance periods apply most fully to the species tested, long leaf pine and Douglas fir, although with failure occurring at the bearings the species of timber is not a governing consideration.

(1) UNPROTECTED TIMBER COLUMNS.—As based on test Nos. 79, 81, 82, and 83, unprotected timber columns with exposed cast iron or steel caps give *25-minute* fire resistance.

(2) PROTECTED TIMBER COLUMNS.—Timber columns having cast iron or steel caps and protected by a layer of gypsum wall board ⅜ inch thick, give *three-fourths hour* fire resistance (test No. 80), and protected by a 1-inch layer of Portland-cement plaster on metal lath, with an air space between the timber and the plaster layer, give *1½-hour* fire resistance (test No. 78).

The protections are assumed to cover all exposed metal of the caps as well as the timber. The proportion of the Portland-cement plaster can be taken the same as given for plaster coverings on steel columns (Table 46, p. 197).

Protections on timber columns should be applied with caution on account of possible unfavorable effects upon the timber in point of susceptibility to decay. (Cf. Sec. XII, p. 194).

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3. CONDITIONS GOVERNING FIRE DURATION IN BUILDINGS

The intensity and duration of building fires depend upon the character and amount of combustible material in the building construction itself and in the building contents. The latter will be determined in a general way by the occupancy. Office and residence occupancies generally support fires of shorter duration than manufacturing, merchandizing, or storage. The possible duration of fire is also determined to some extent by the floor load for which the building is designed, since it limits the amount of material subject to fire.

The effective fire duration, as far as it concerns a given building member, pertains only to the duration of the combustion taking place near enough to it to impart temperatures sufficiently high to cause failure under its supported load. This is not necessarily the same as the total duration of the fire within the building, or within a subdivision thereof, such as a building story. It is necessary to assume maximum probable conditions both with regard to building contents and air supply, as considered with respect to intensity and duration of a possible fire. Compensations and adjustments between intensity and duration may be necessary under some conditions in order to approximate a fire duration having intensity equivalent to that of the exposure in the fire test. Limitation of the degree of resistance to be developed to the requirements of present or immediate future occupancy is generally not justified, unless the building, its location or use is such that no change to a use involving a more severe fire condition is probable.

While it is outside of the scope of the present paper to attempt the determination of probable fire duration for various types of buildings and occupancies, it is a step in the application of the test results that will require careful consideration. The fire-resistance periods were derived under exacting conditions in point of test columns, test conditions, and interpretation of results, and the types of columns and protections to which they pertain are deemed to be fully adequate for resisting building fires of corresponding duration.

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XIV. APPENDIXES

APPENDIX A.—Views of Columns

Fig. No.	Test No.	Protection	Page
		BEFORE AND AFTER FIRE TEST	
58	1 to 5	Unprotected structural-steel columns	217
59	6 to 8	do	218
60	9 to 13	Unprotected cast-iron and pipe columns	219
61	14 to 18	Partly protected structural-steel columns	220
62	19 to 22	do	221
63	23 to 27	Plaster on metal-lath protections	222
64	28A to 31	Concrete protections	223
65	32 to 34A	do	224
66	35 to 39, 45	do	225
67	40 to 44, 46, 47	do	226
68	48, 49, 50	Clay-tile protections	227
69	50A, 51, 51A	do	228
70	52, 53, 54	do	229
71	55 to 58	do	230
72	59, 60	do	231
73	61, 62, 63	do	232
74	64, 65, 66	Gypsum-block protections	233
75	67, 67A	do	234
76	68, 69	Brick protections	235
76	70	Reinforced-concrete column	235
77	71 to 75	Reinforced-concrete columns	236
78	76, 77	Plastered clay-tile protections	237
79	78	Longleaf pine column, protected	238
80	79	Longleaf pine column, unprotected	239
81	80	Longleaf pine column, protected	240
81	81	Longleaf pine column, unprotected	240
82	82, 83	Douglas fir columns, unprotected	241
		BEFORE AND AFTER FIRE AND WATER TEST	
83	101, 102, 103	Concrete protections	242
84	104	Concrete protection	243
84	105	Clay-tile protection	243
85	106, 107	Clay-tile protections	244
86	108, 109	Gypsum-block protections	245
87	110	Plaster on metal-lath protection	246
88	111, 112	Reinforced-concrete columns	247
89	113	Reinforced-concrete column	248
89	114, 115	Cast-iron columns	248

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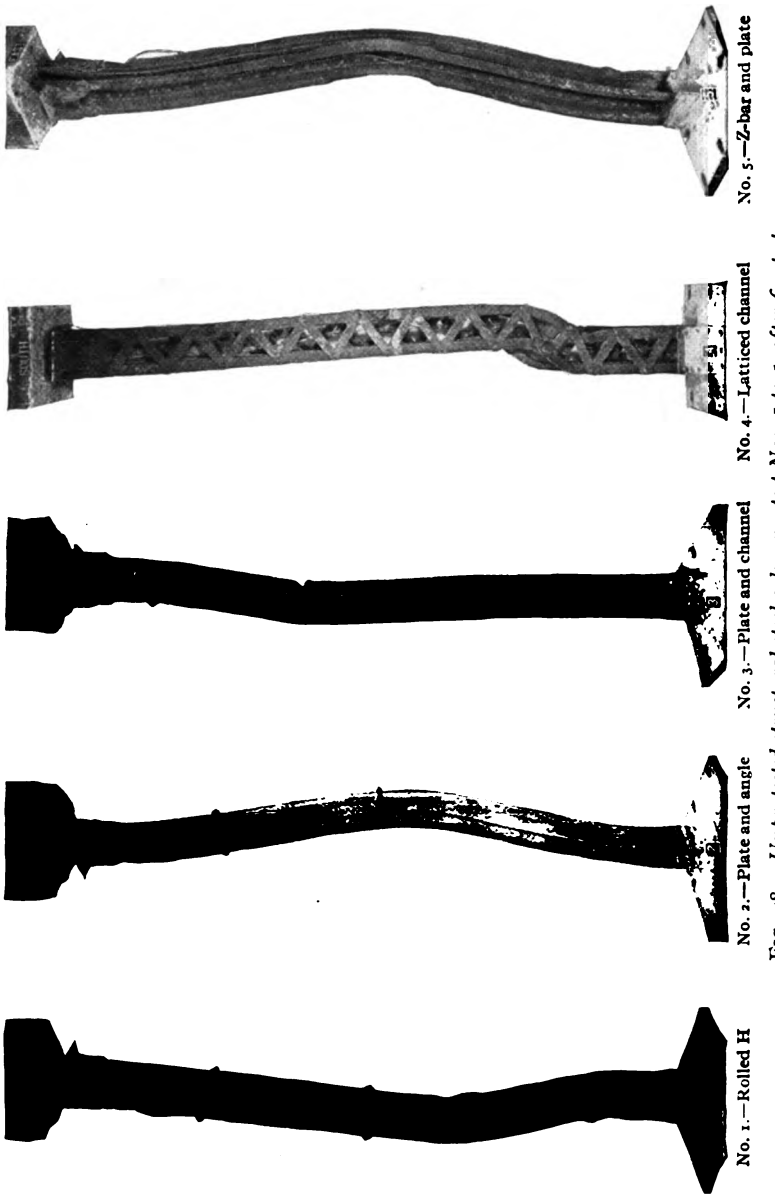

FIG. 58.—*Unprotected structural-steel columns, test Nos. 1 to 5, after fire test*

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No. 6.—I-beam and channel No. 7.—Latticed angle No. 7.—Detail of failure No. 8.—Starred angle

FIG. 59.—*Unprotected structural-steel columns, test Nos. 6 to 8, after fire test*

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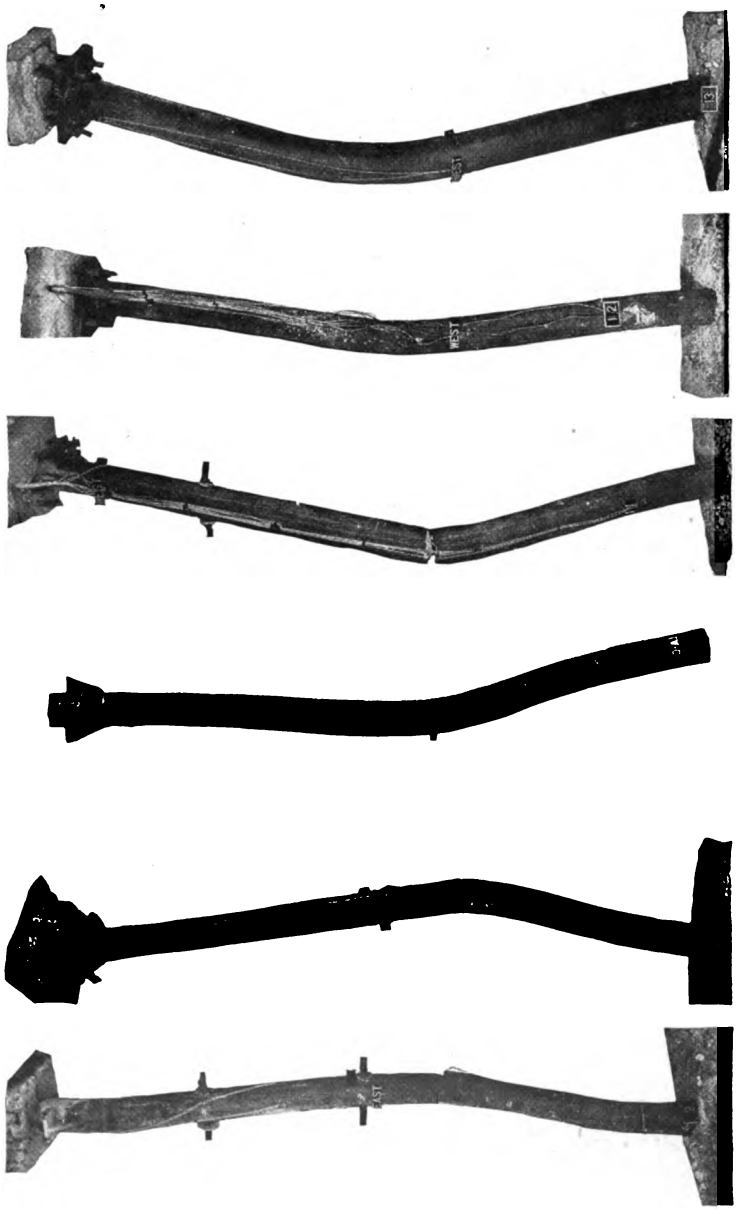
No. 9.—Cast iron, ends restrained
No. 10.—Cast iron, ends not restrained
No. 10A.—Cast-iron pipe
No. 11.—Cast iron, concrete filled
No. 12.—Steel pipe, concrete filled
No. 13.—Reinforced-steel pipe, concrete filled

FIG. 60.—*Unprotected cast iron and pipe columns, test Nos. 9 to 13, after fire test*

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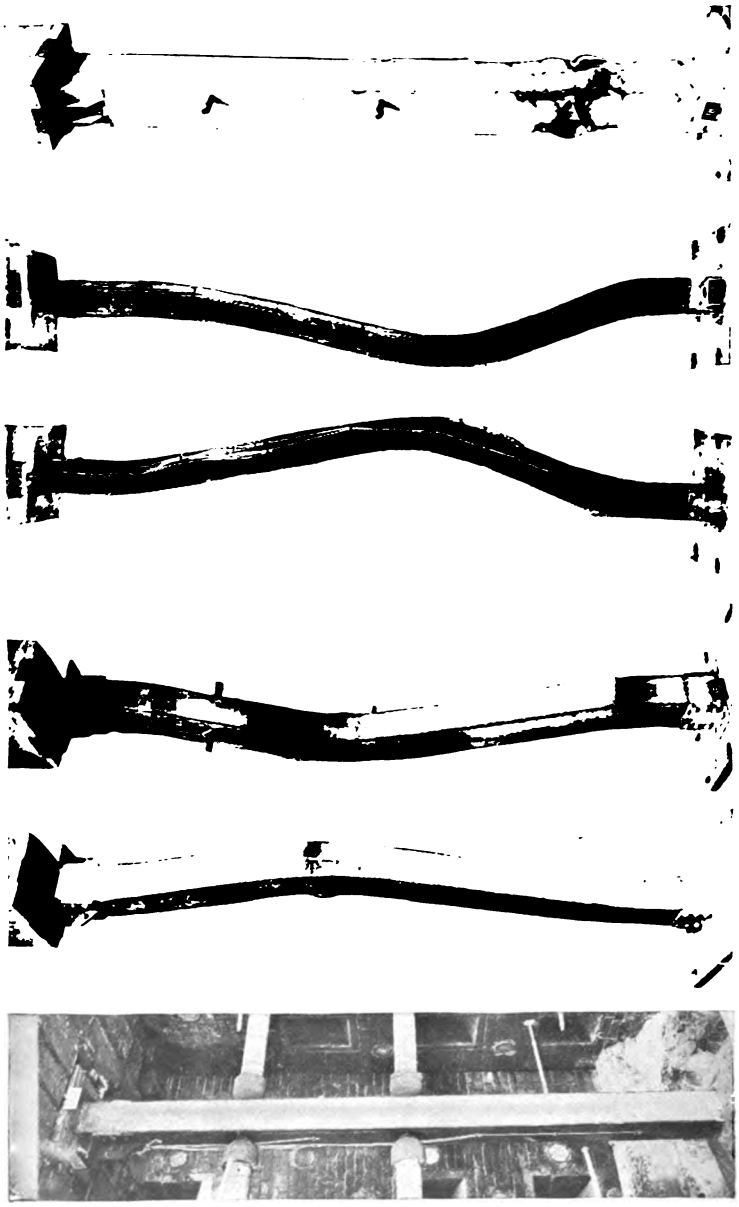

FIG. 61.—*Partly protected structural-steel columns, test Nos. 14 to 18, after fire test*

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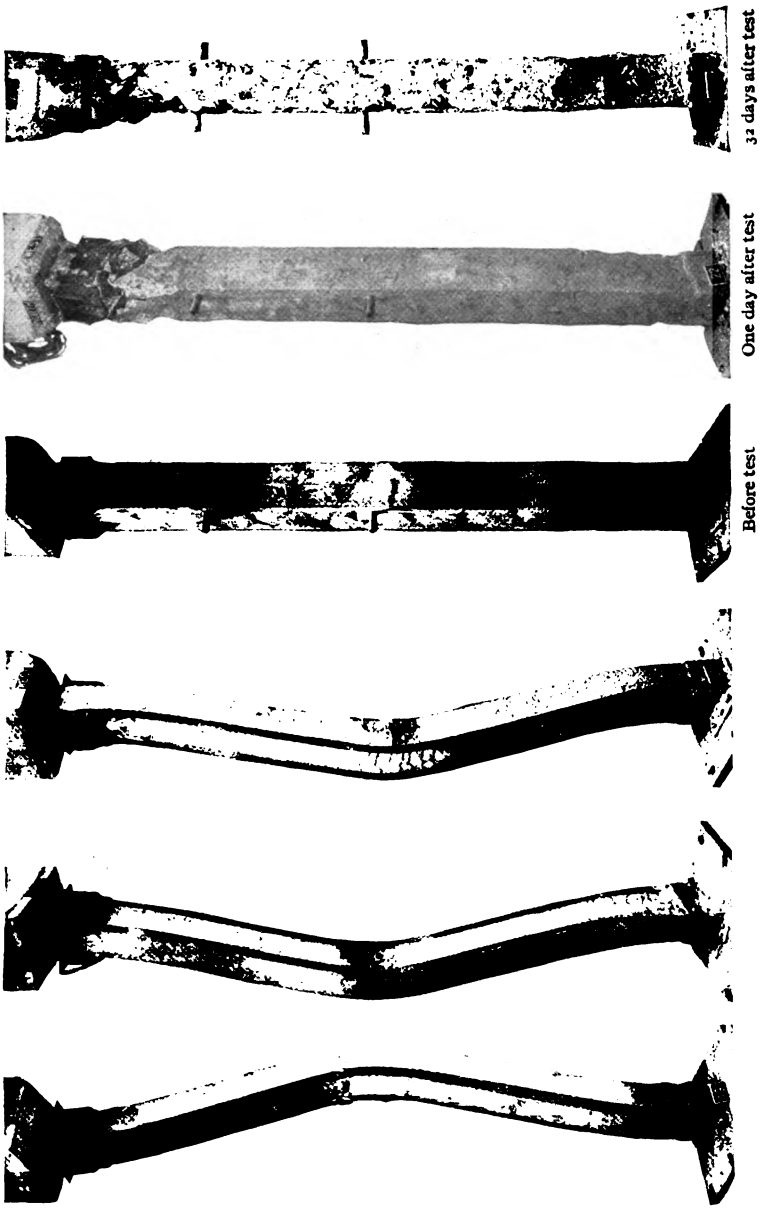

FIG. 62.—*Partly protected structural-steel columns, test Nos. 19 to 22, after fire test*

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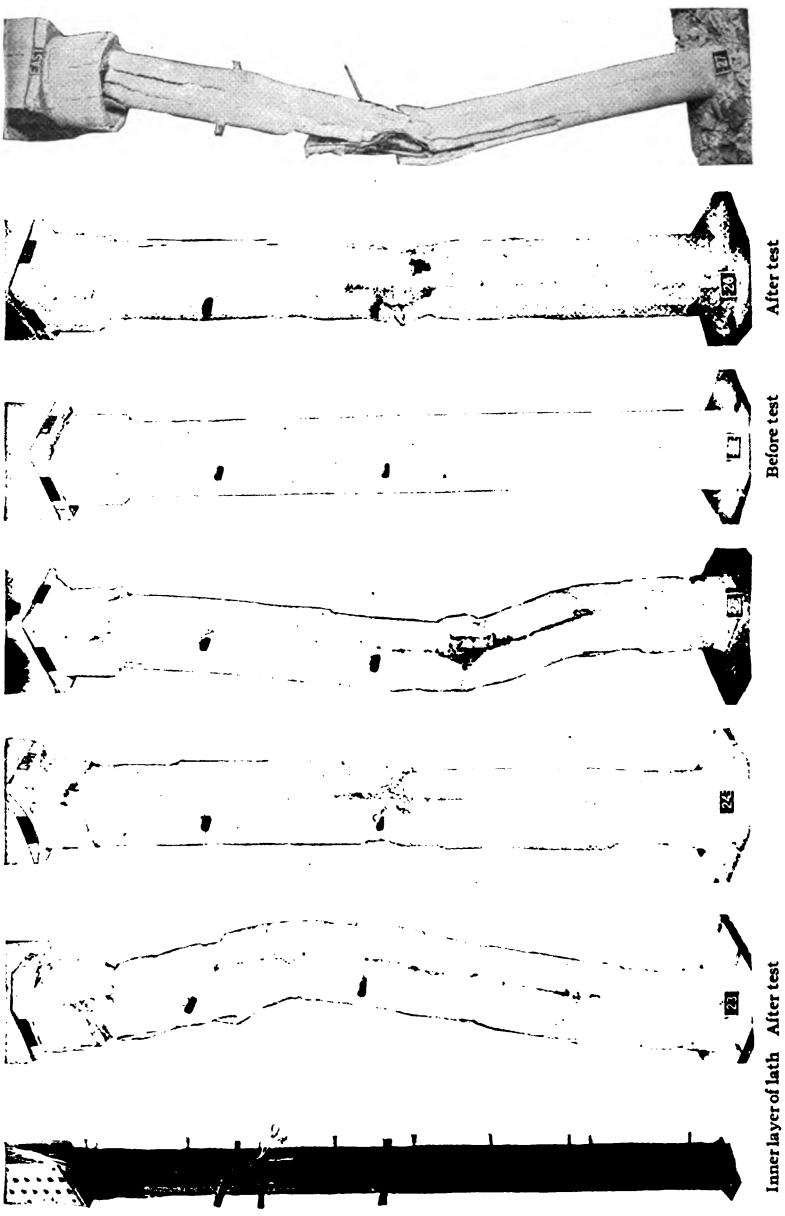

FIG. 63.—*Plaster on metal-lath protections, test Nos. 23 to 27, after fire test*

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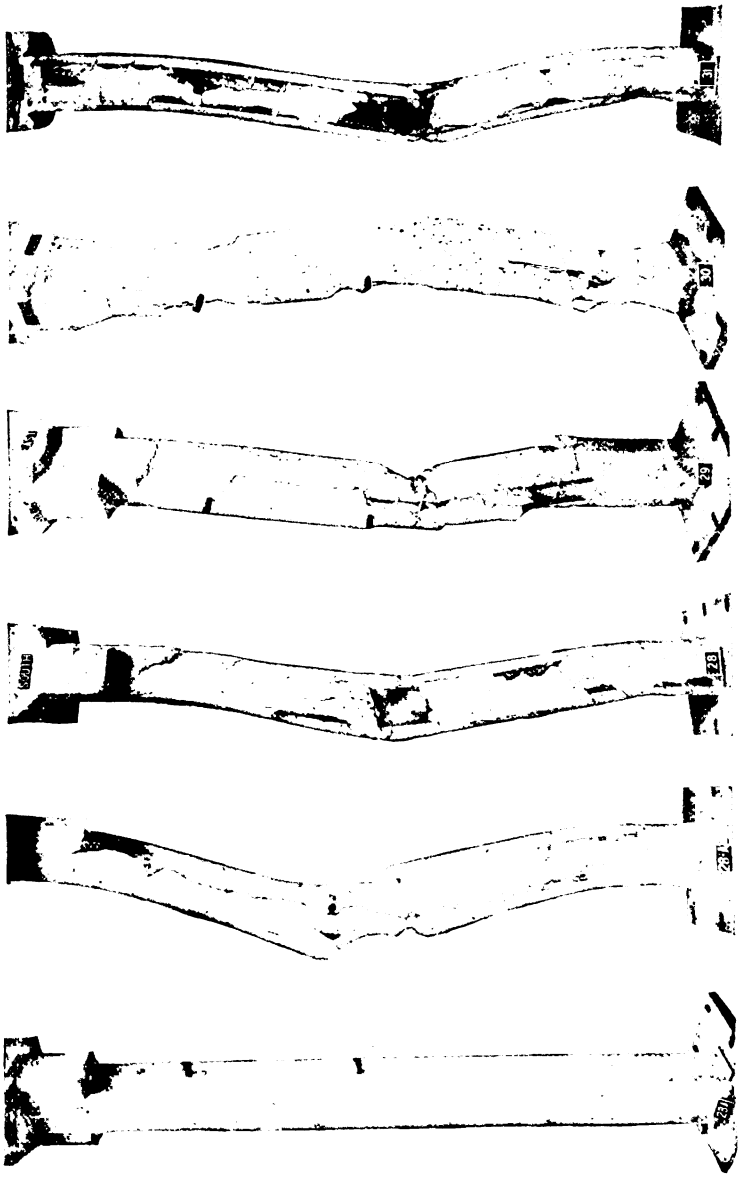

FIG. 64.—*Concrete protections, test Nos. 28A to 31, after fire test*

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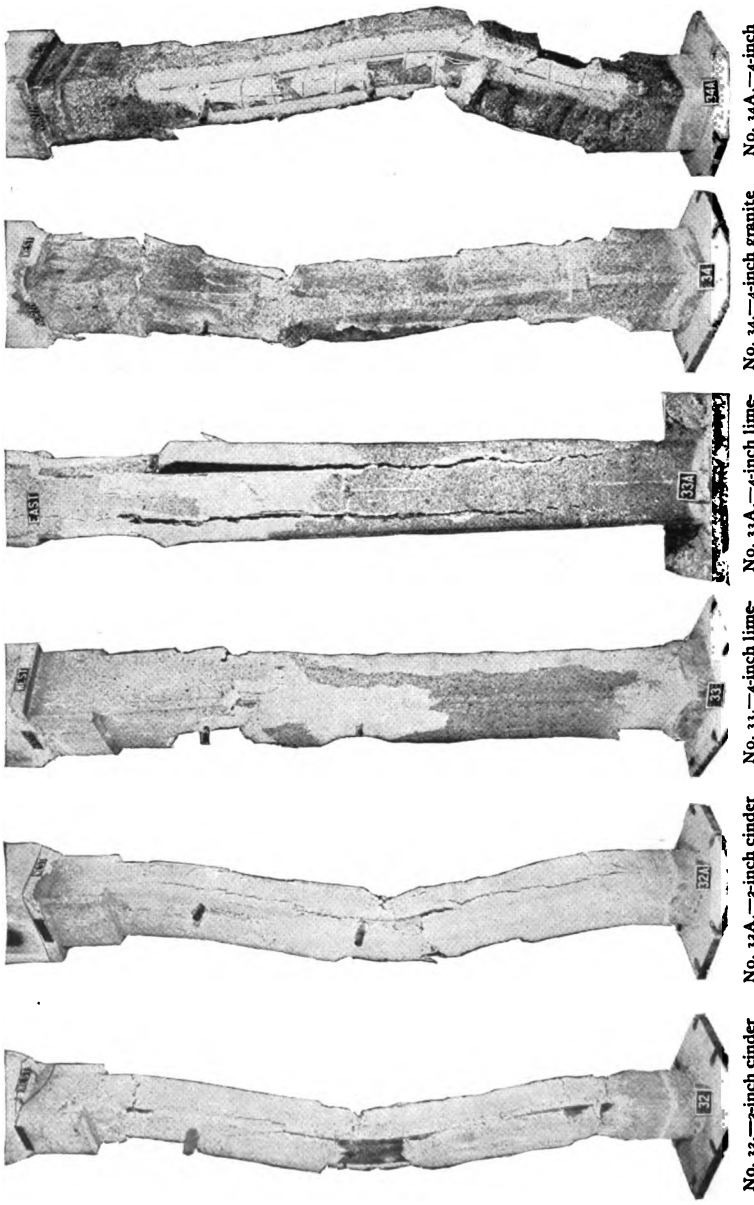

FIG. 65—*Concrete prolections, test Nos. 32 to 34A, after fire test*

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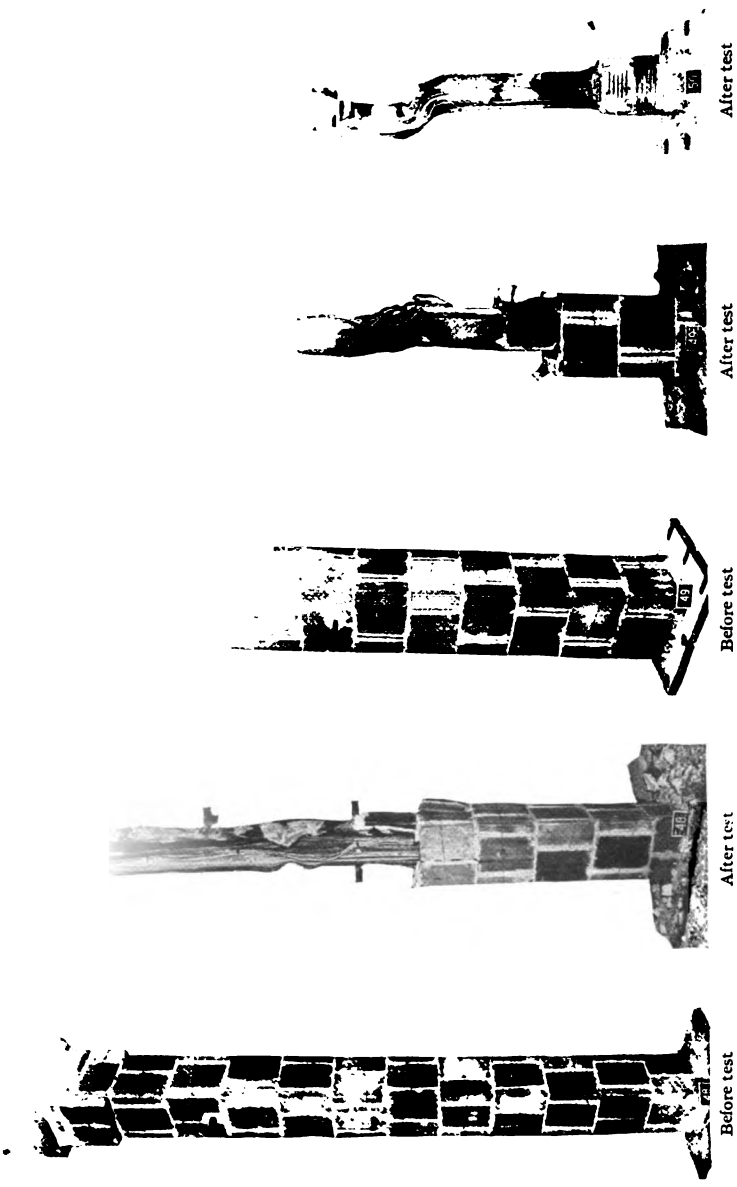

FIG. 68.—*Clay-tile protections, test Nos. 48, 49, and 50, before and after fire test*

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FIG. 67.—*Concrete protections, test Nos. 40 to 44, 46, and 47, after fire test*

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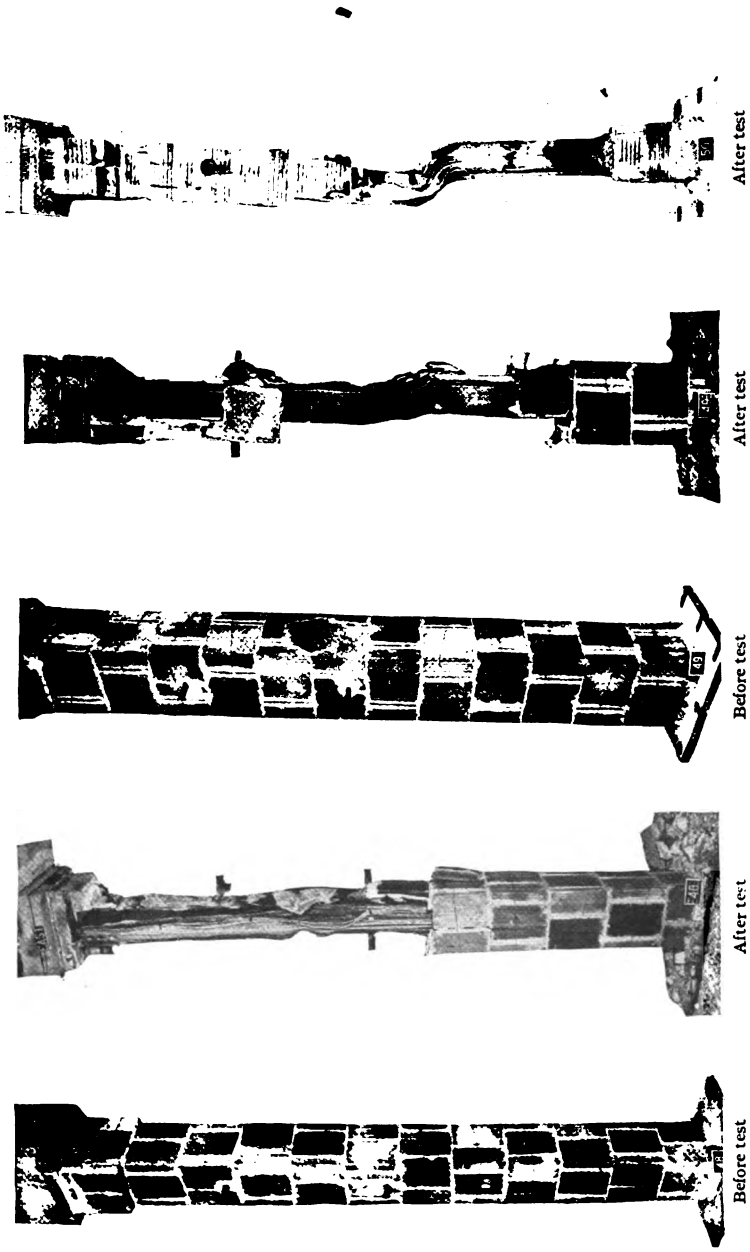

FIG. 68.—*Clay-tile protections, test Nos. 48, 49, and 50, before and after fire test*

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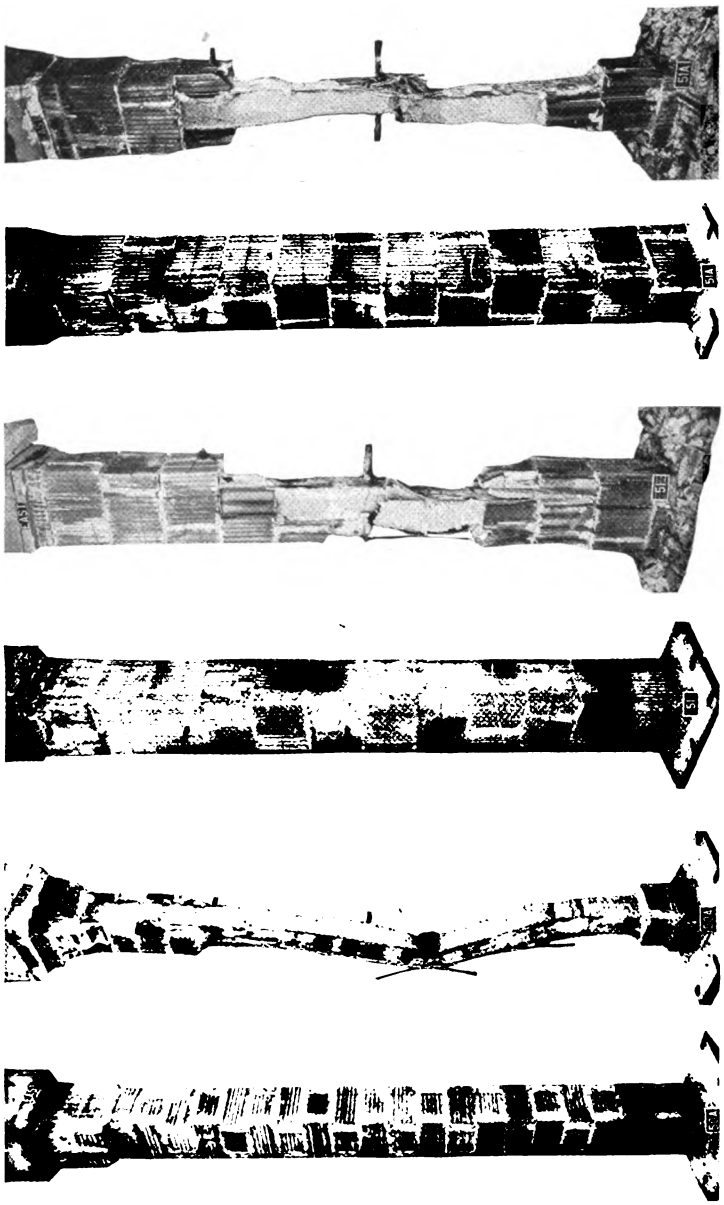

FIG. 69.—*Clay-tile protections, test Nos. 50A, 51, and 51A, before and after fire test*

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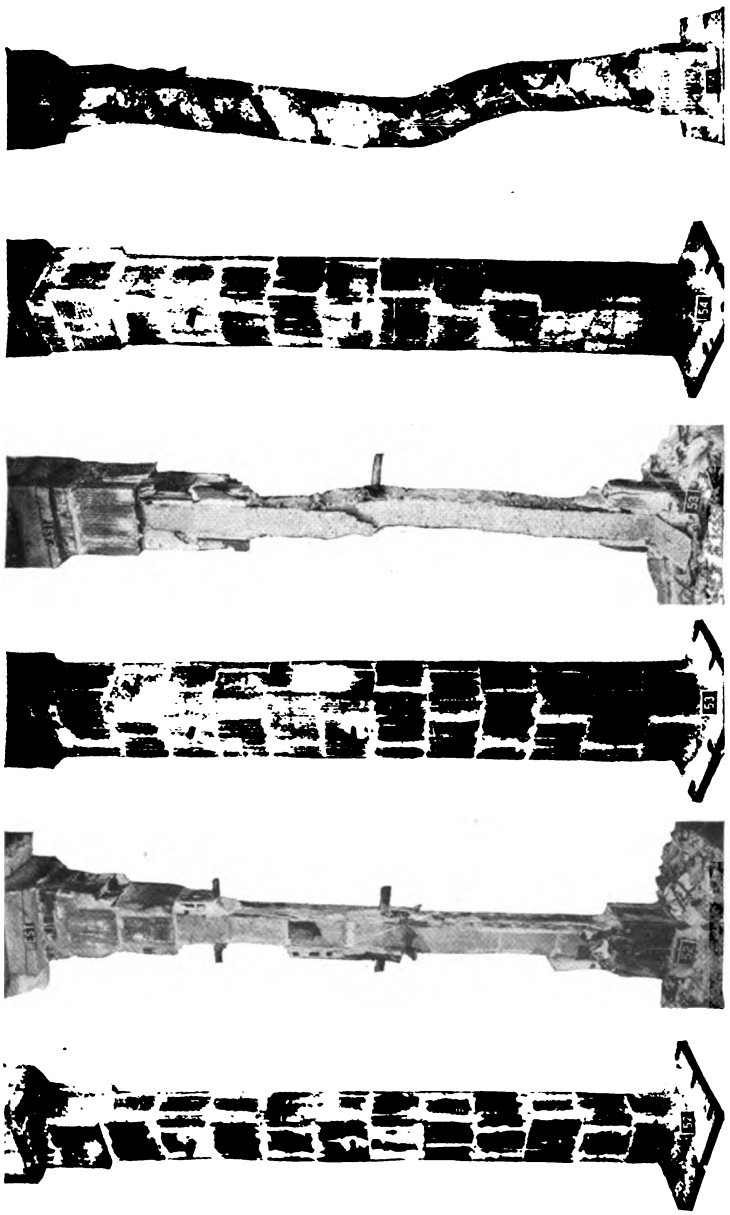

FIG. 70.—*Clay-tile protections, test Nos. 52, 53, and 54, before and after fire test*

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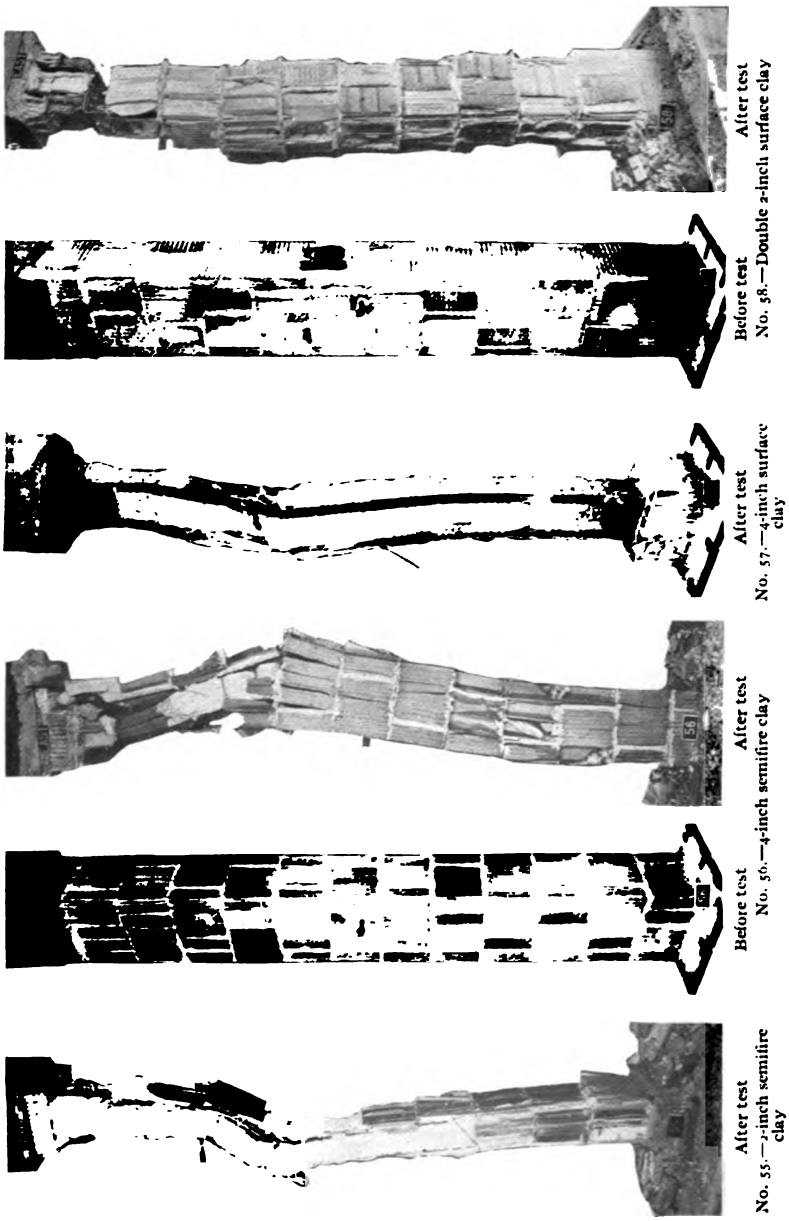

FIG. 71.—*Clay-tile protections, test Nos. 55 to 58, before and after fire test*

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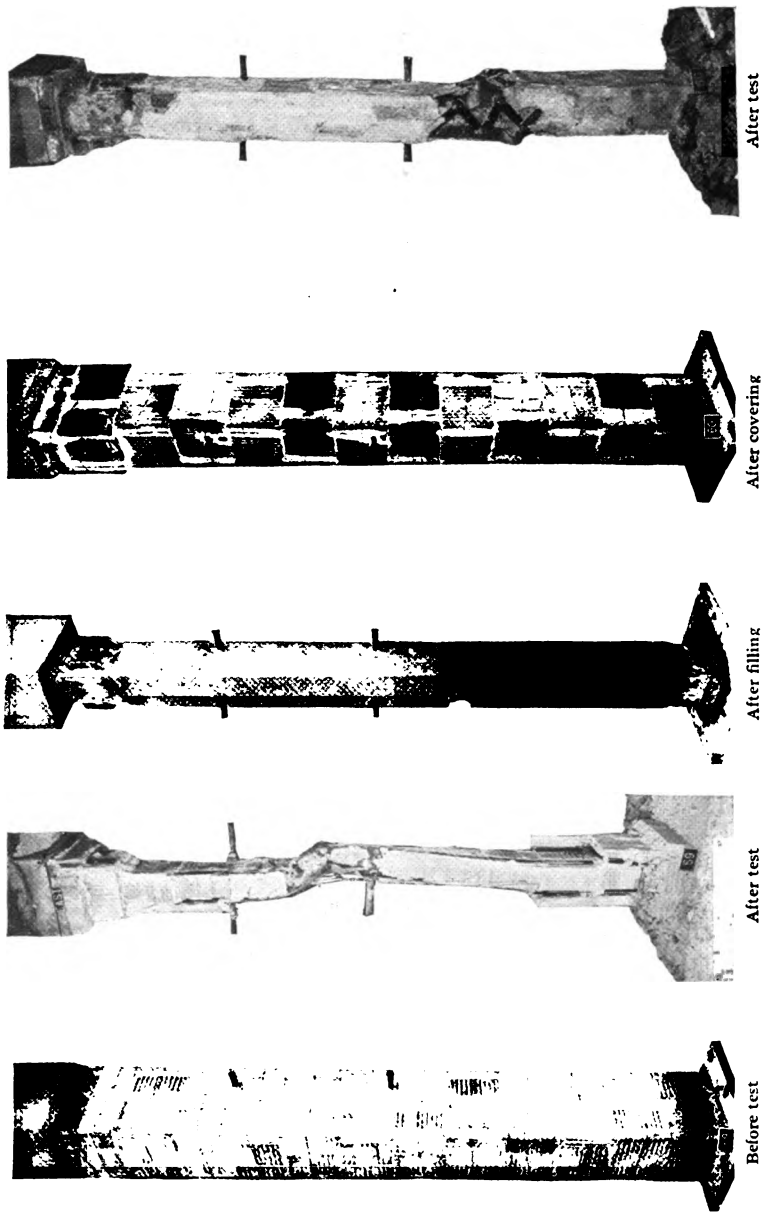

FIG. 72.—*Clay-tile protections, test Nos. 59 and 60, before and after fire test*

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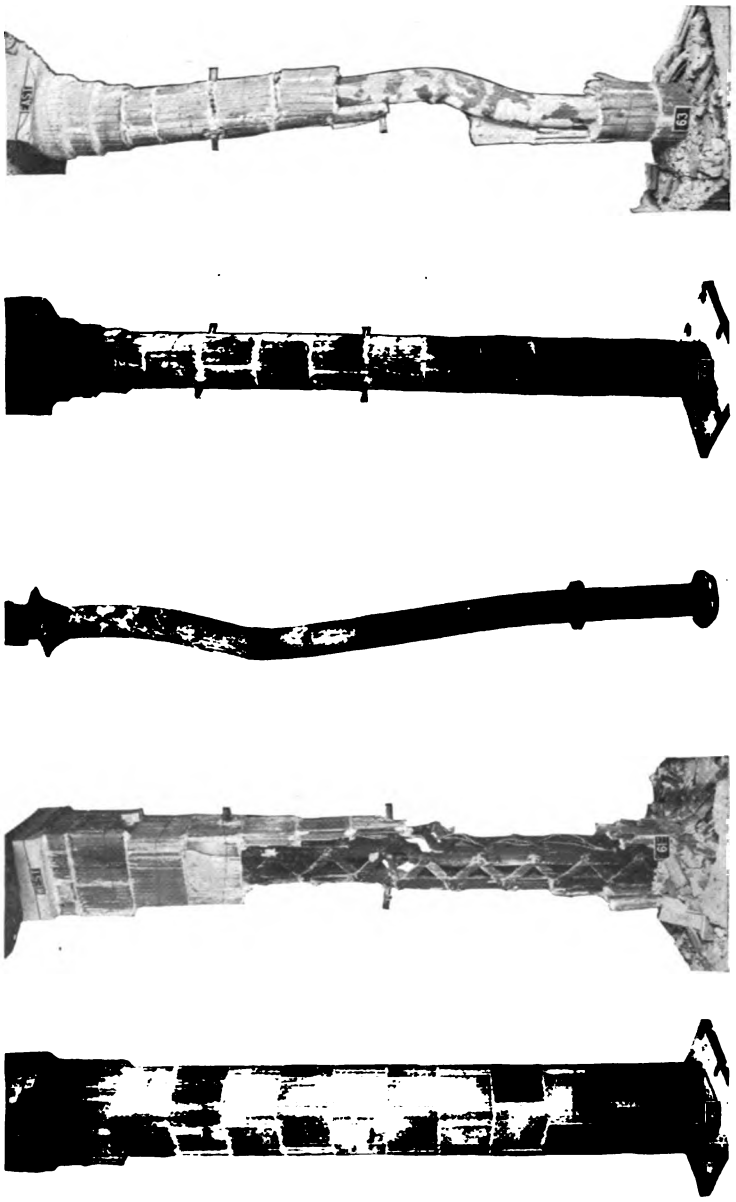

FIG. 73.—*Clay-tile protections, test Nos. 61, 62, and 63, before and after fire test*

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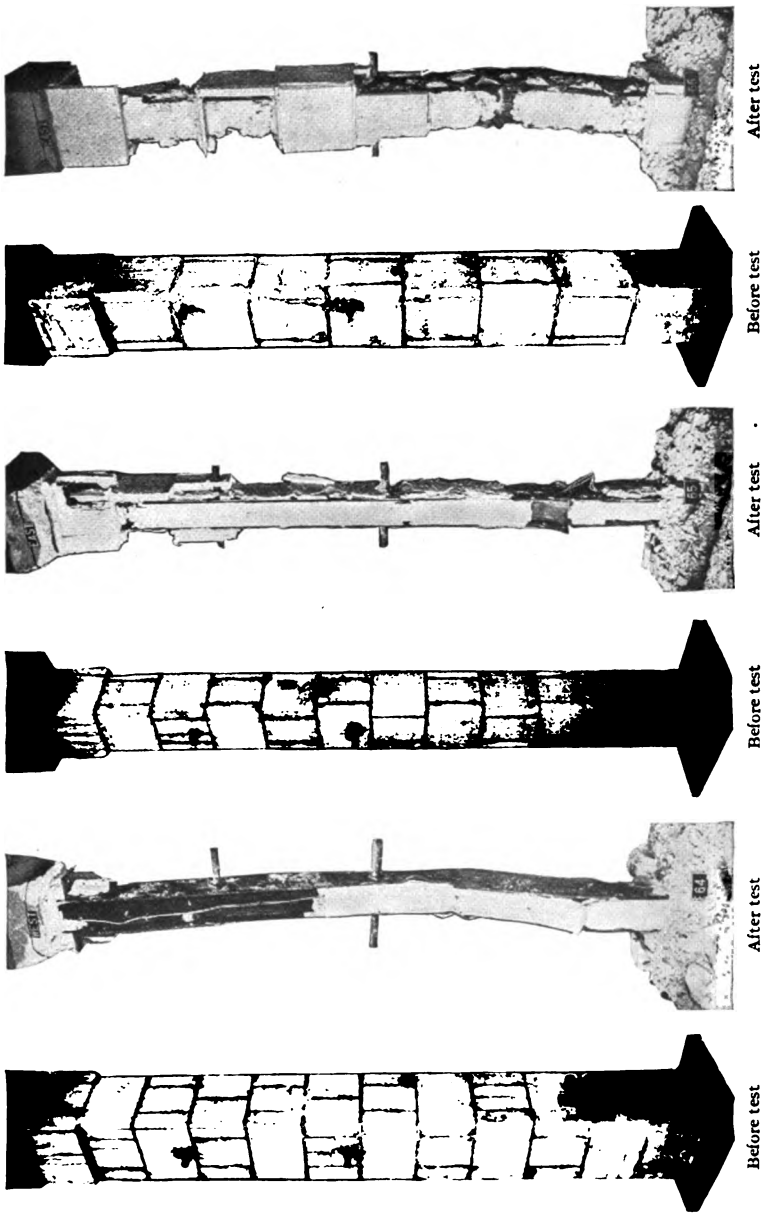

FIG. 74.—*Gypsum-block protections, test Nos. 64, 65, and 66, before and after fire test*

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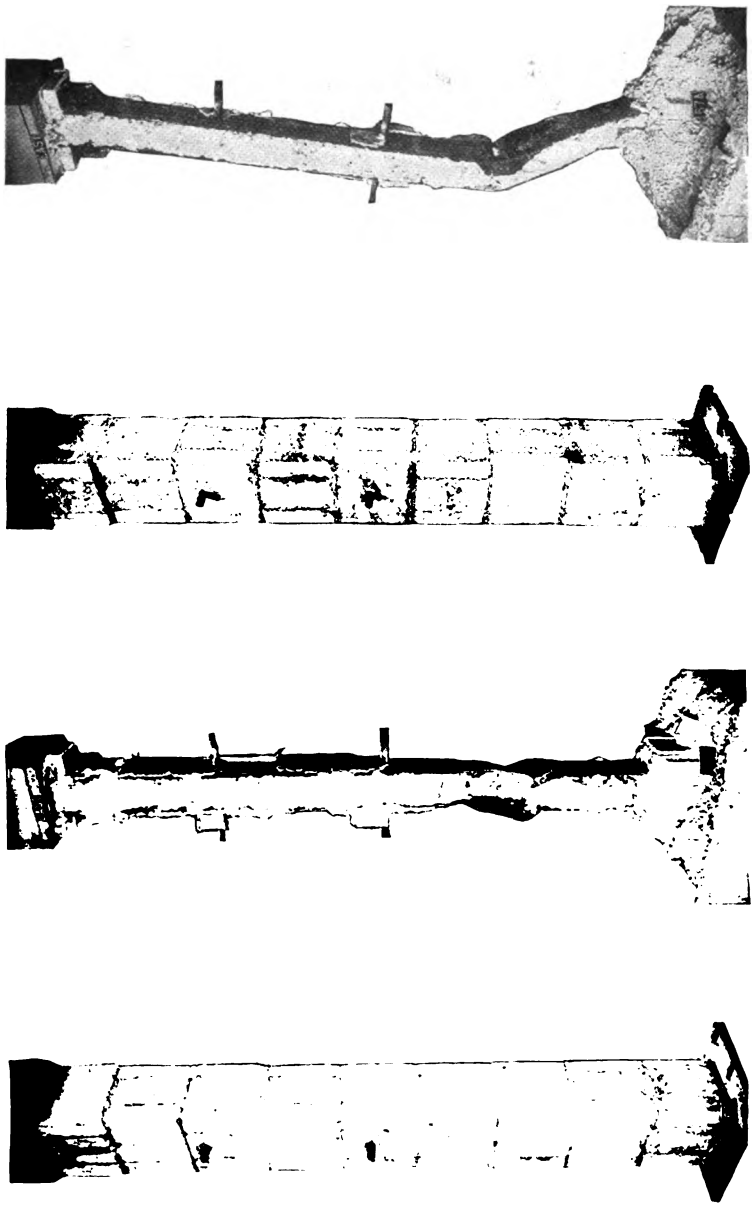

FIG. 75.—*Gypsum-block protections, test Nos. 67 and 67A, before and after fire test*

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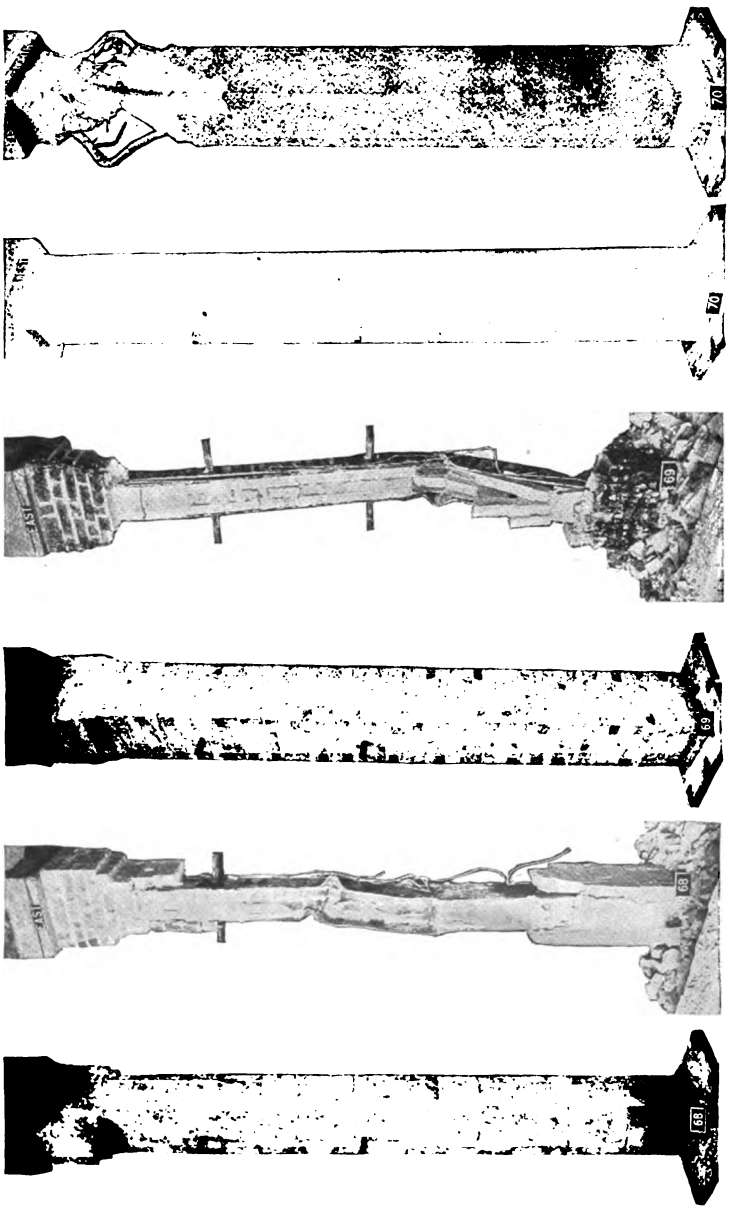

FIG. 76.—*Brick protections, test Nos. 68 and 69; reinforced-concrete column, test No. 70, before and after fire test*

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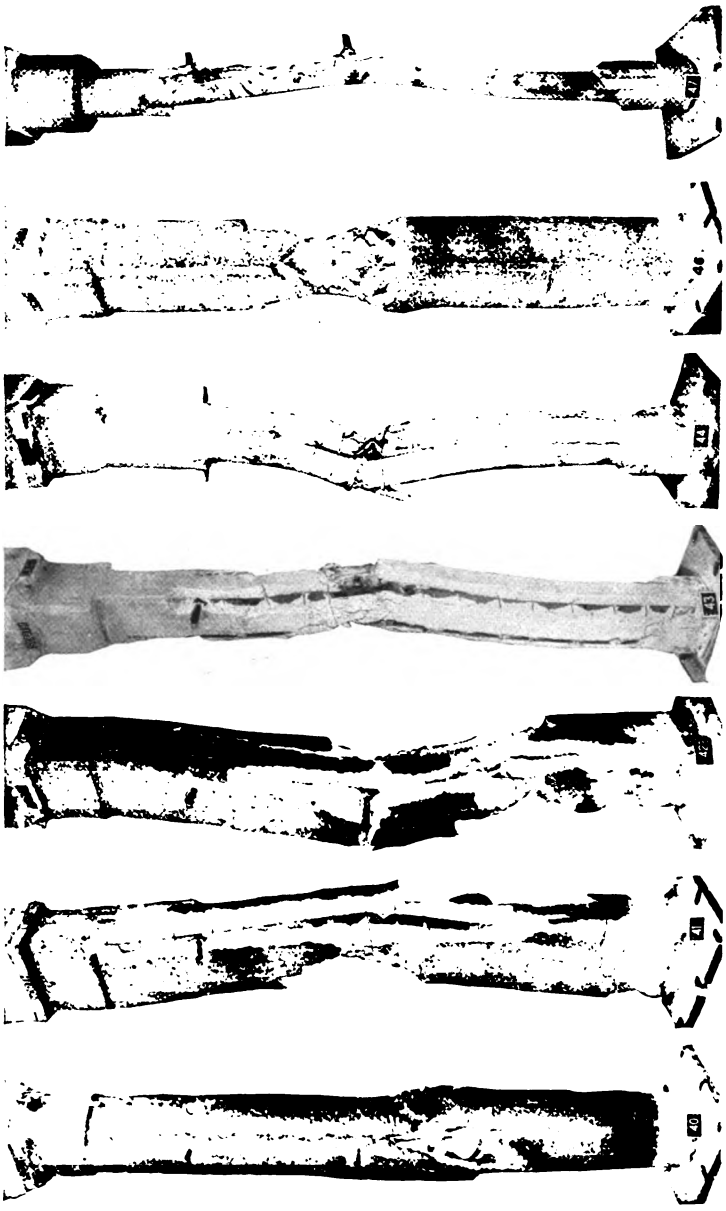

FIG. 67.—*Concrete protections, test Nos. 40 to 44, 46, and 47, after fire test*

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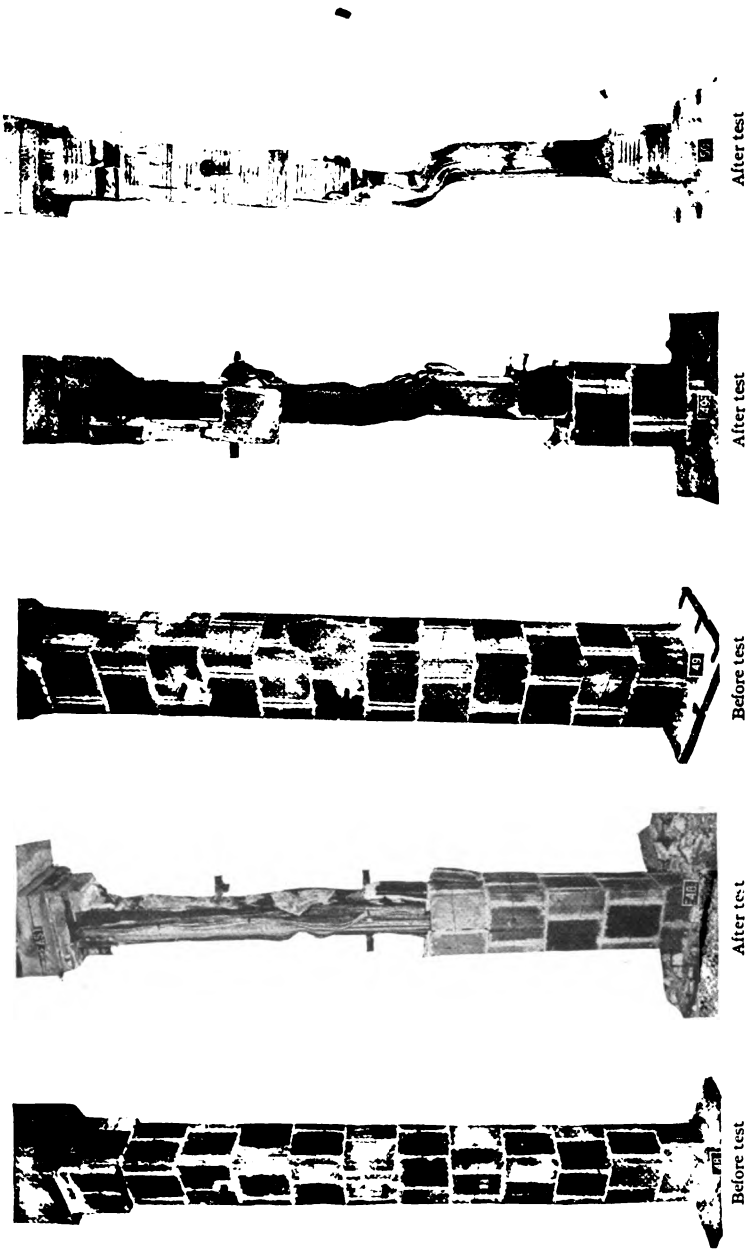

FIG. 68.—*Clay-tile protections, test Nos. 48, 49, and 50, before and after fire test*

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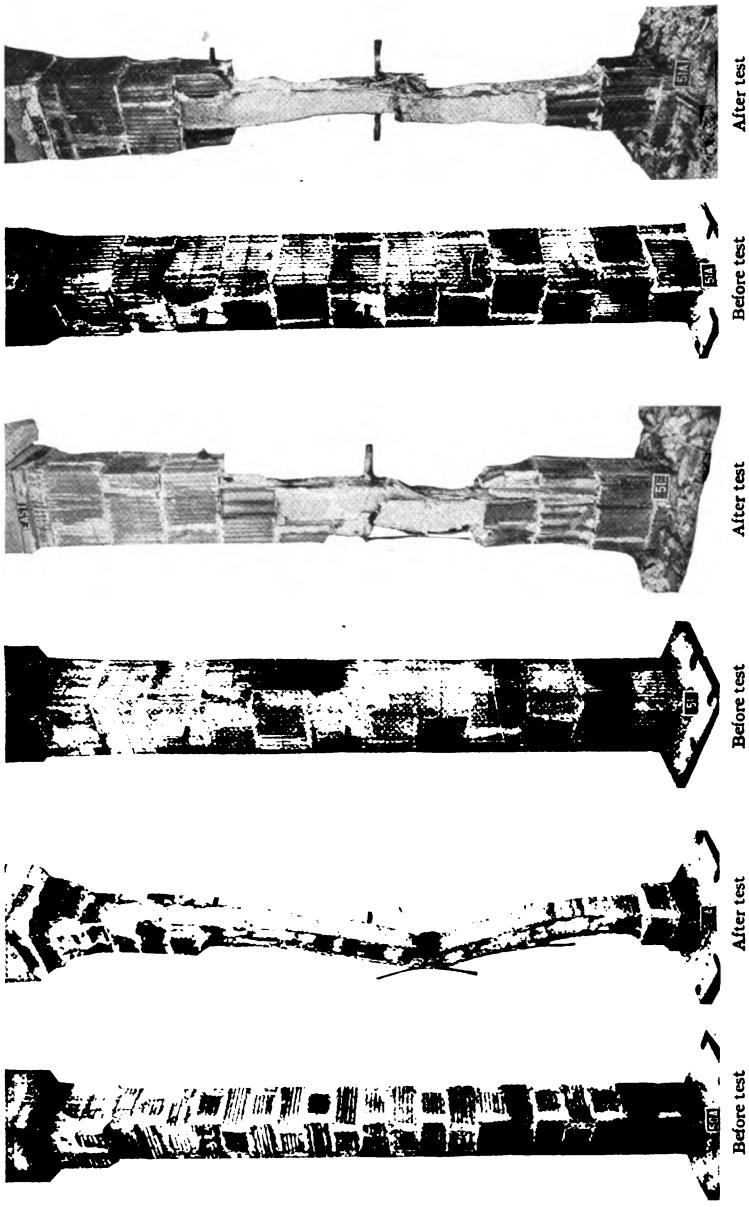

FIG. 69.—*Clay-tile protections, test Nos. 50A, 51, and 51A, before and after fire test*

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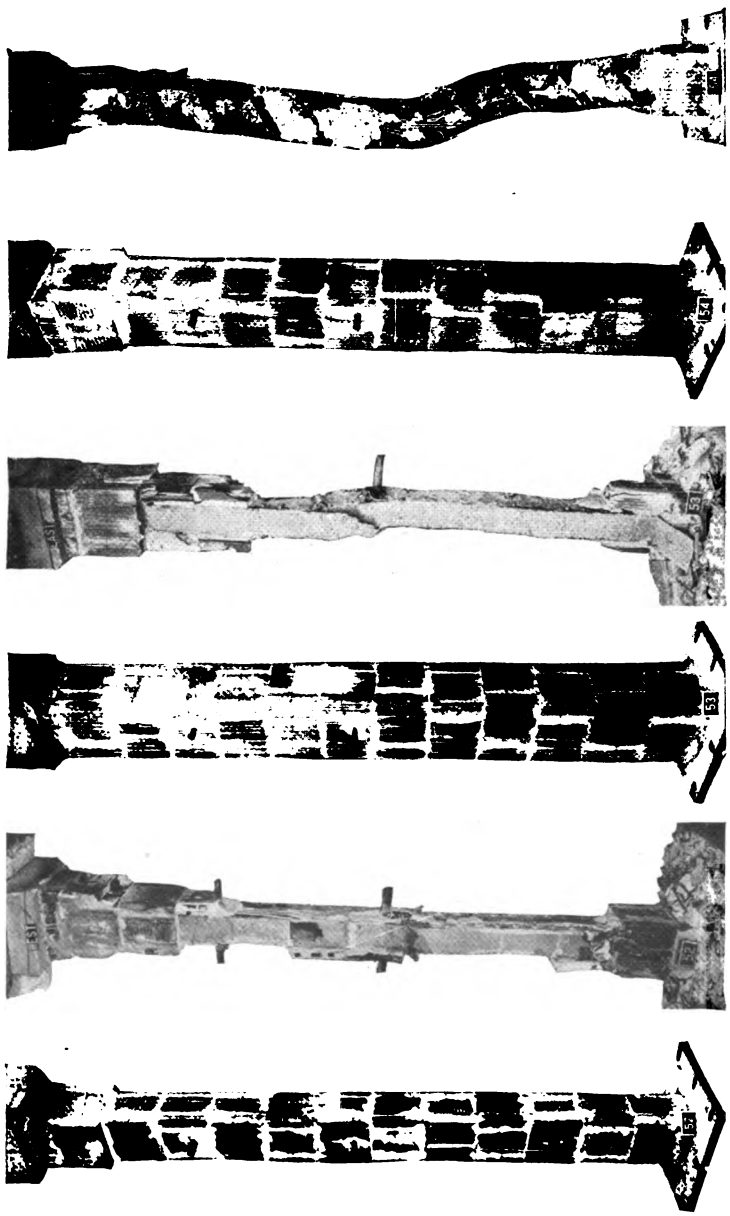

FIG. 70.—*Clay-tile protections, test Nos. 52, 53, and 54, before and after fire test*

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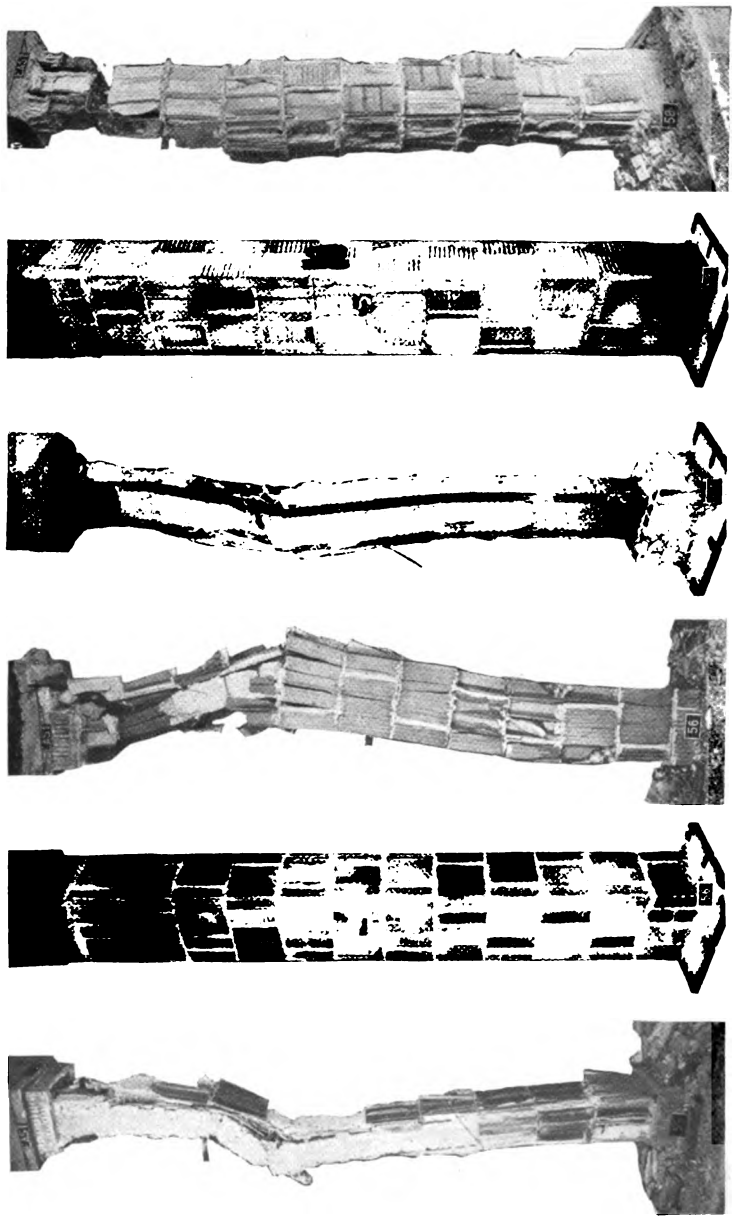

FIG. 71.—*Clay-tile protections, test Nos. 55 to 58, before and after fire test*

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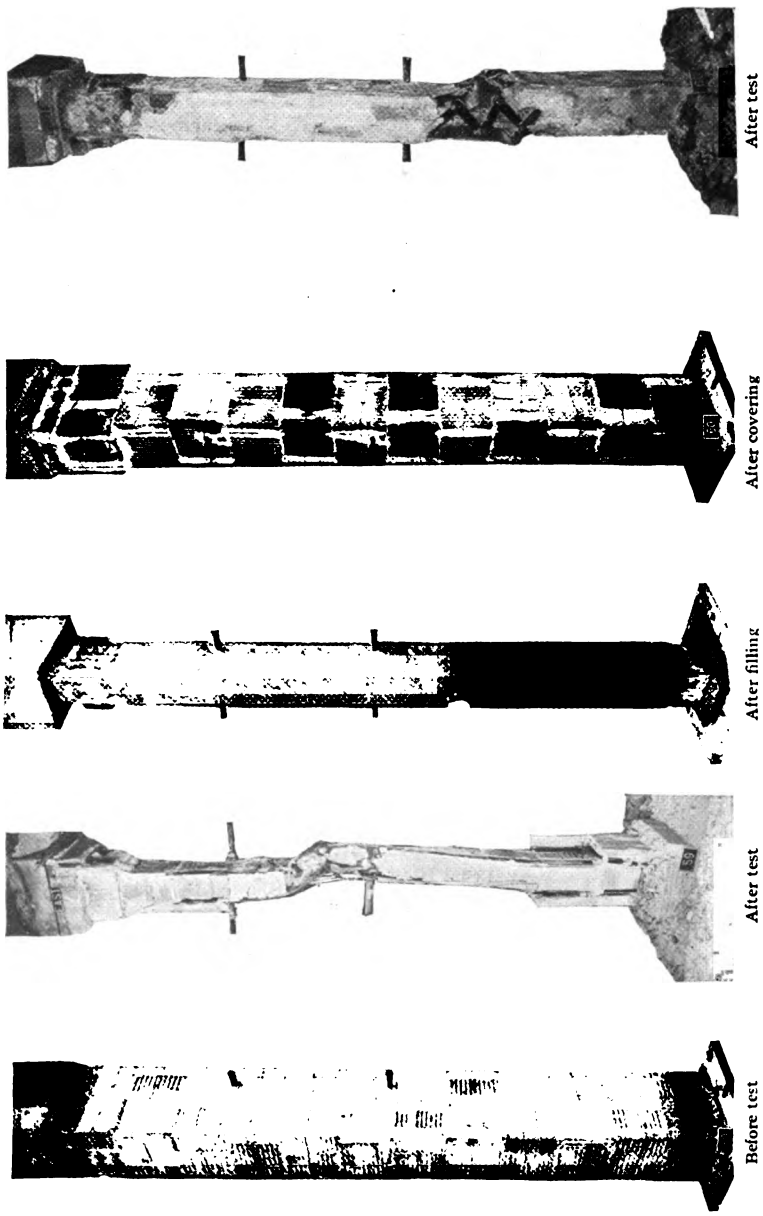

FIG. 72.—*Clay-tile protections, test Nos. 59 and 60, before and after fire test*

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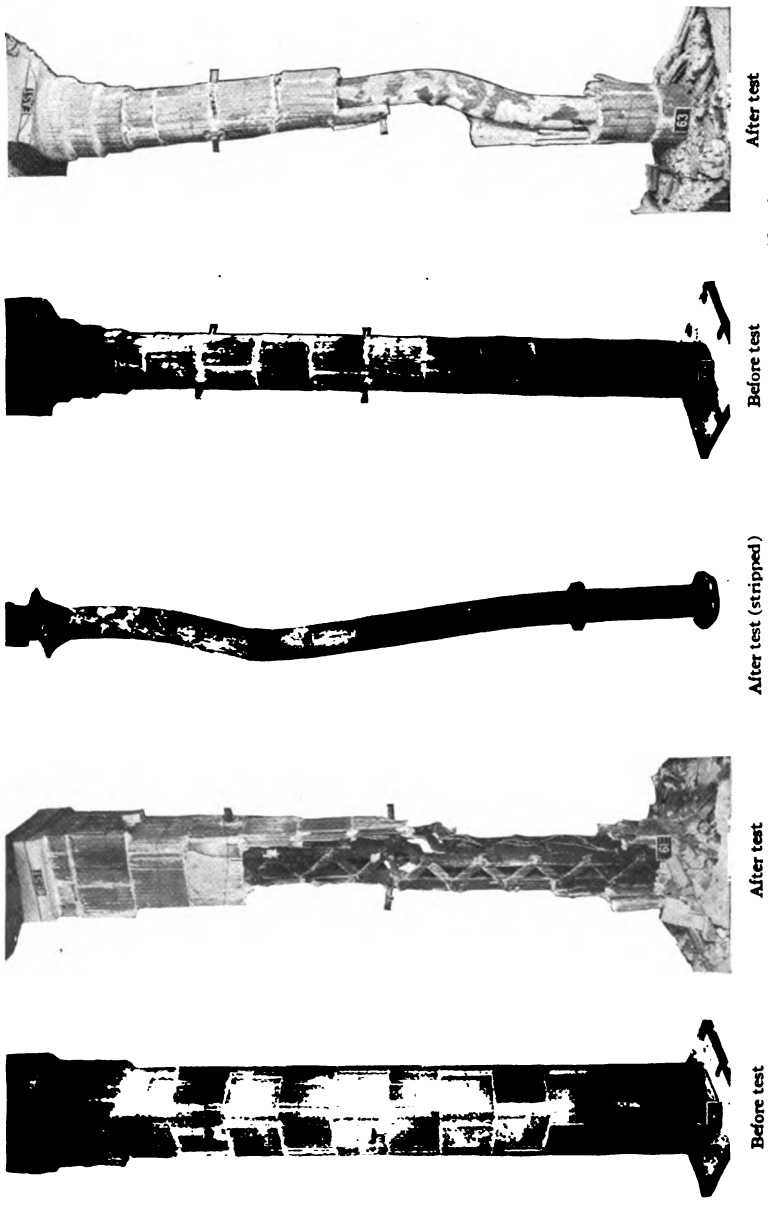

FIG. 73.—*Clay-tile protections, test Nos. 61, 62, and 63, before and after fire test*

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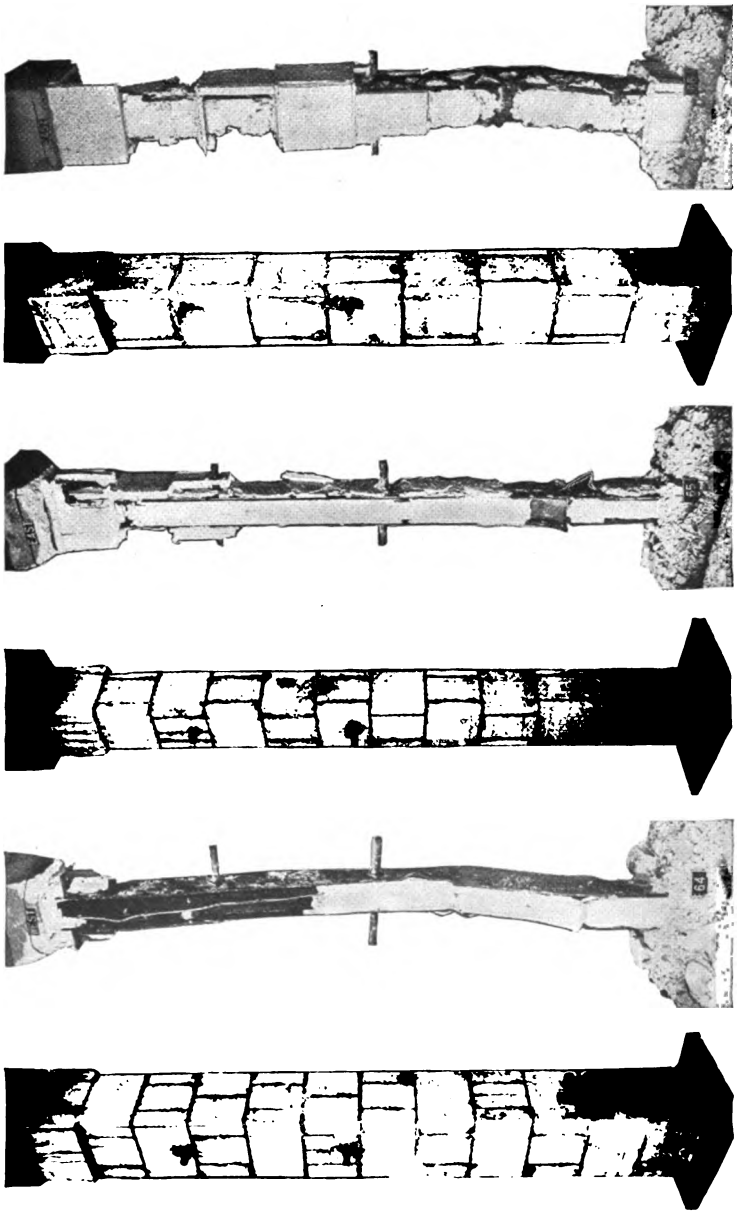

FIG. 74.—*Gypsum-block protections, test Nos. 64, 65, and 66, before and after fire test*

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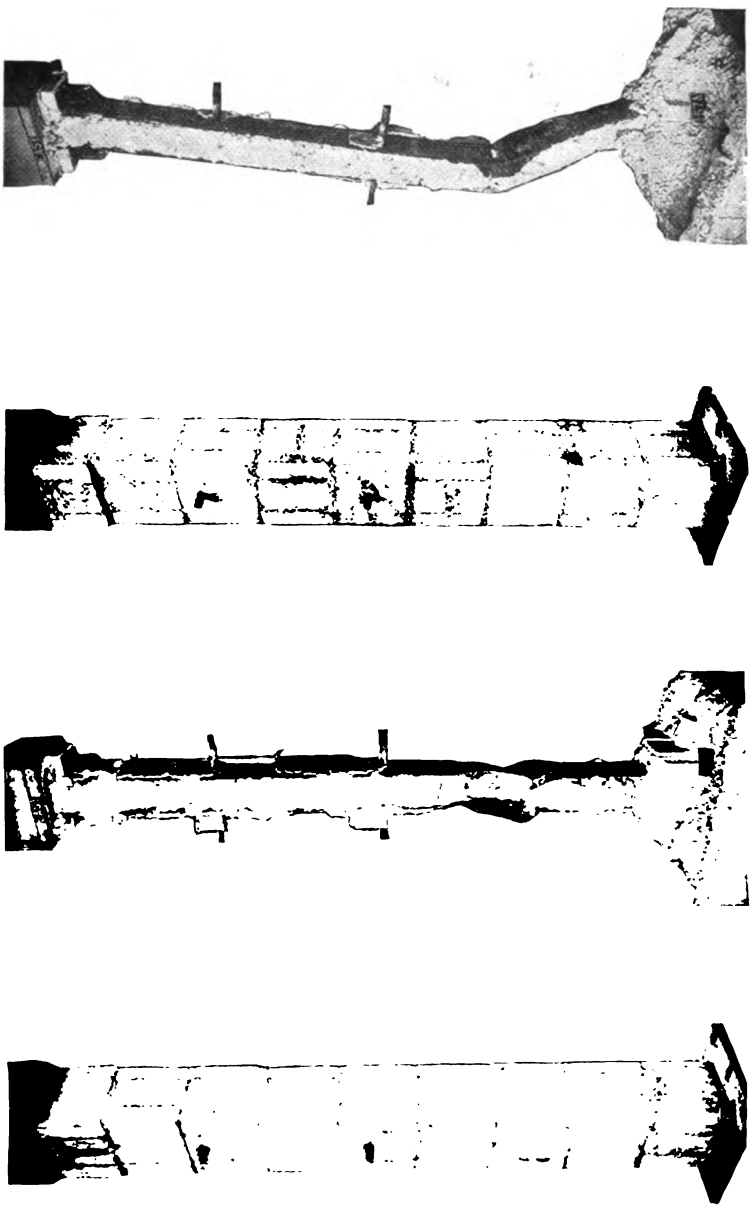

FIG. 75.—*Gypsum-block protections, test Nos. 67 and 67A, before and after fire test*

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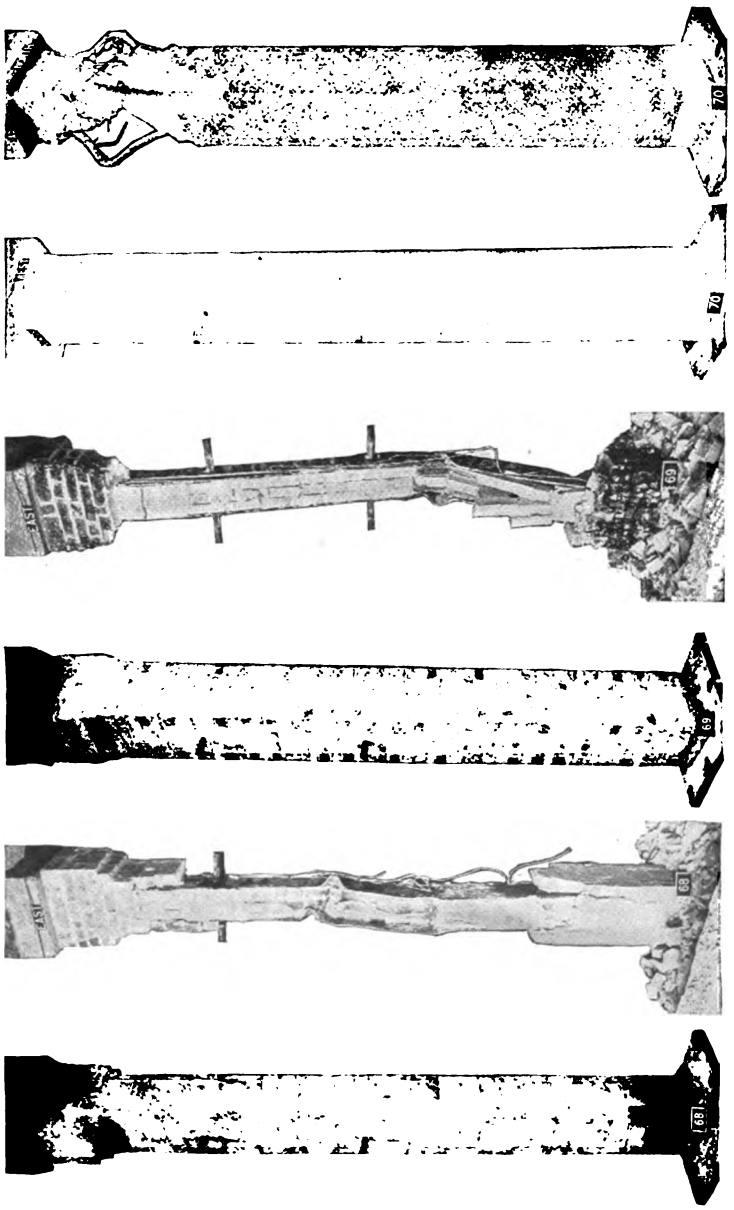

FIG. 76.—*Brick protections, test Nos. 68 and 69; reinforced-concrete column, test No. 70, before and after fire test*

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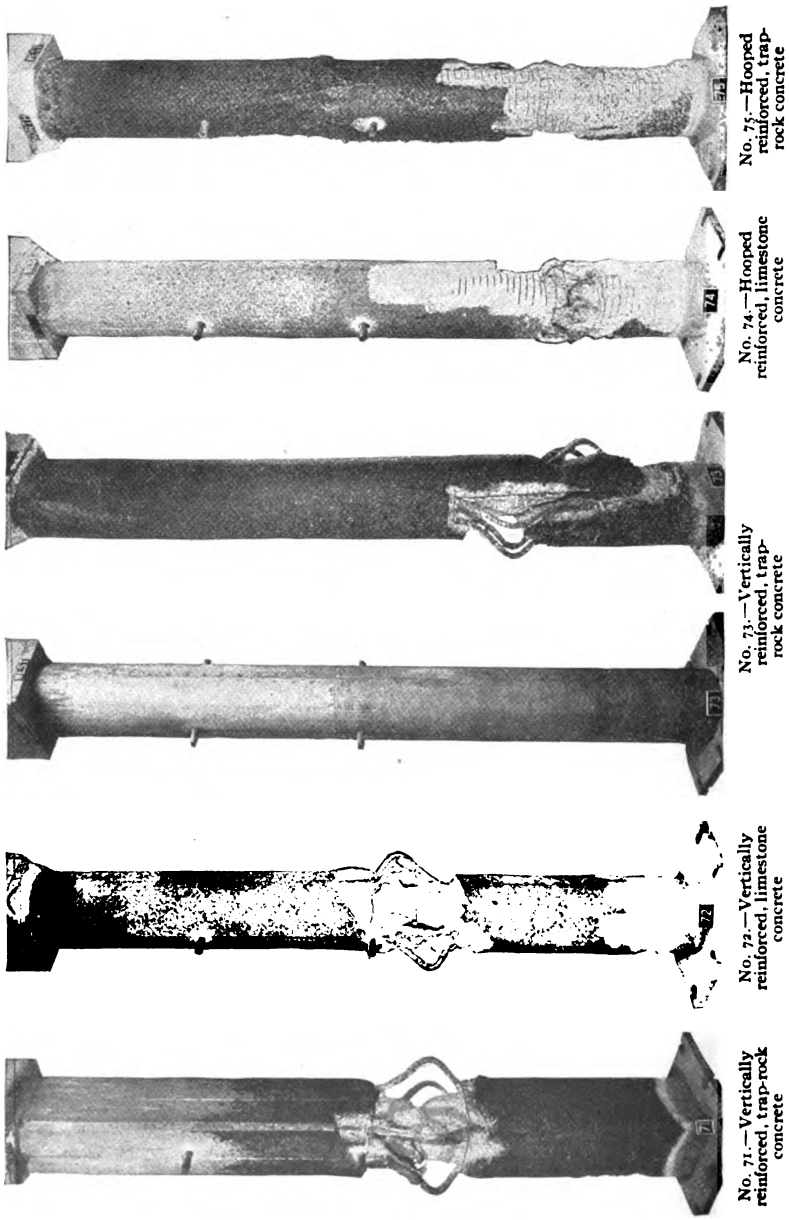
No. 71.—Vertically reinforced, trap-rock concrete
No. 72.—Vertically reinforced, limestone concrete
No. 73.—Vertically reinforced, trap-rock concrete
No. 74.—Hooped reinforced, limestone concrete
No. 75.—Hooped reinforced, trap-rock concrete

FIG. 77.—*Reinforced-concrete columns, test Nos. 71 to 75, after fire test*

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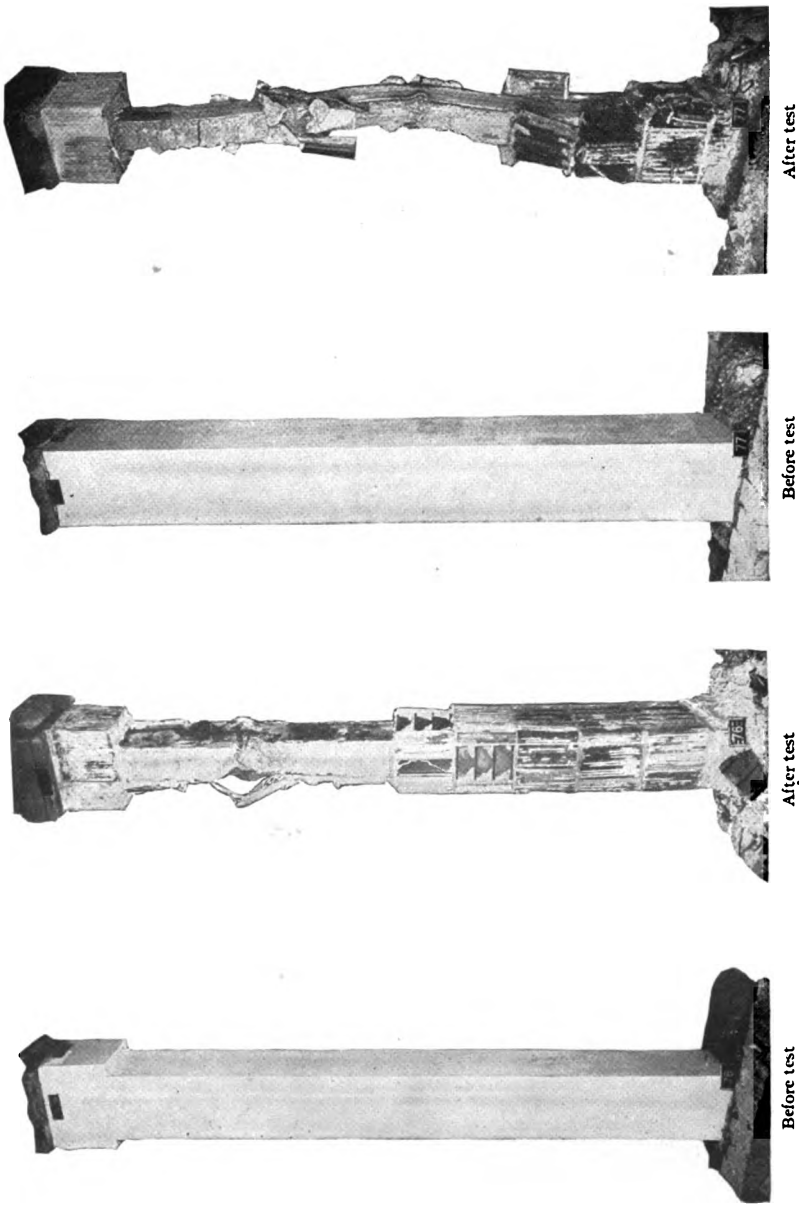

FIG. 78.—*Plastered clay-tile protections, test Nos. 76 and 77, before and after fire test*

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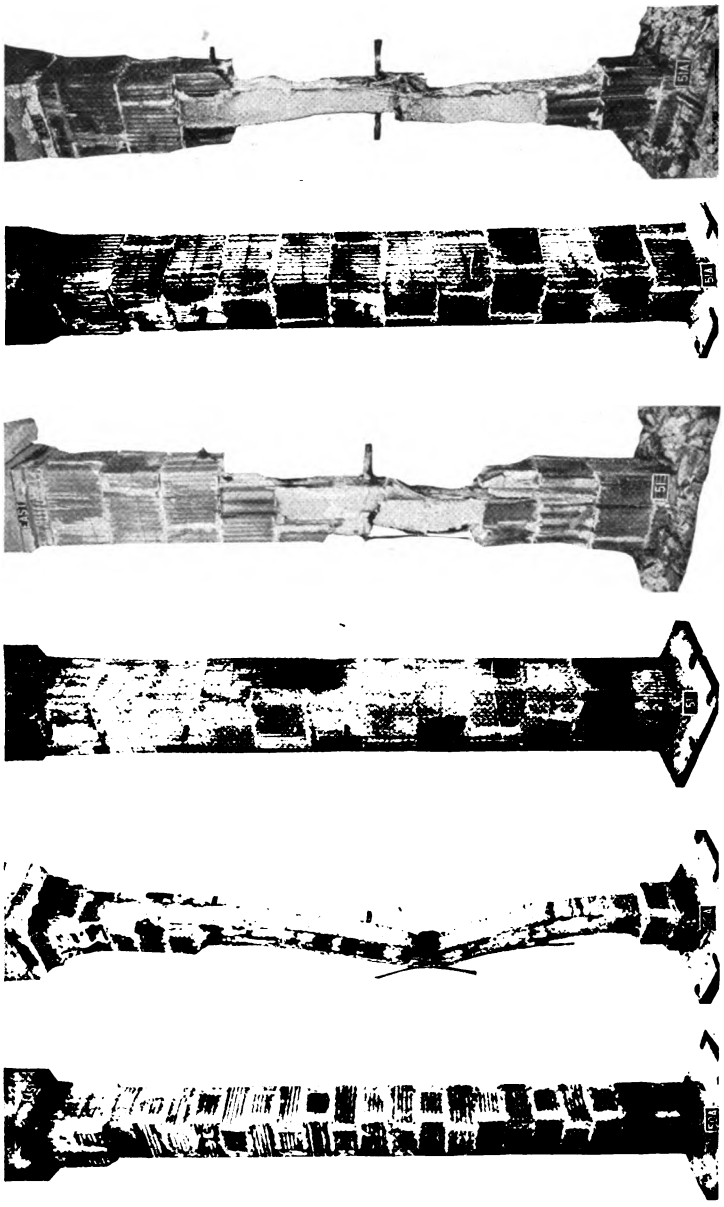

FIG. 69.—*Clay-tile projections, test Nos. 50A, 51, and 51A, before and after fire test*

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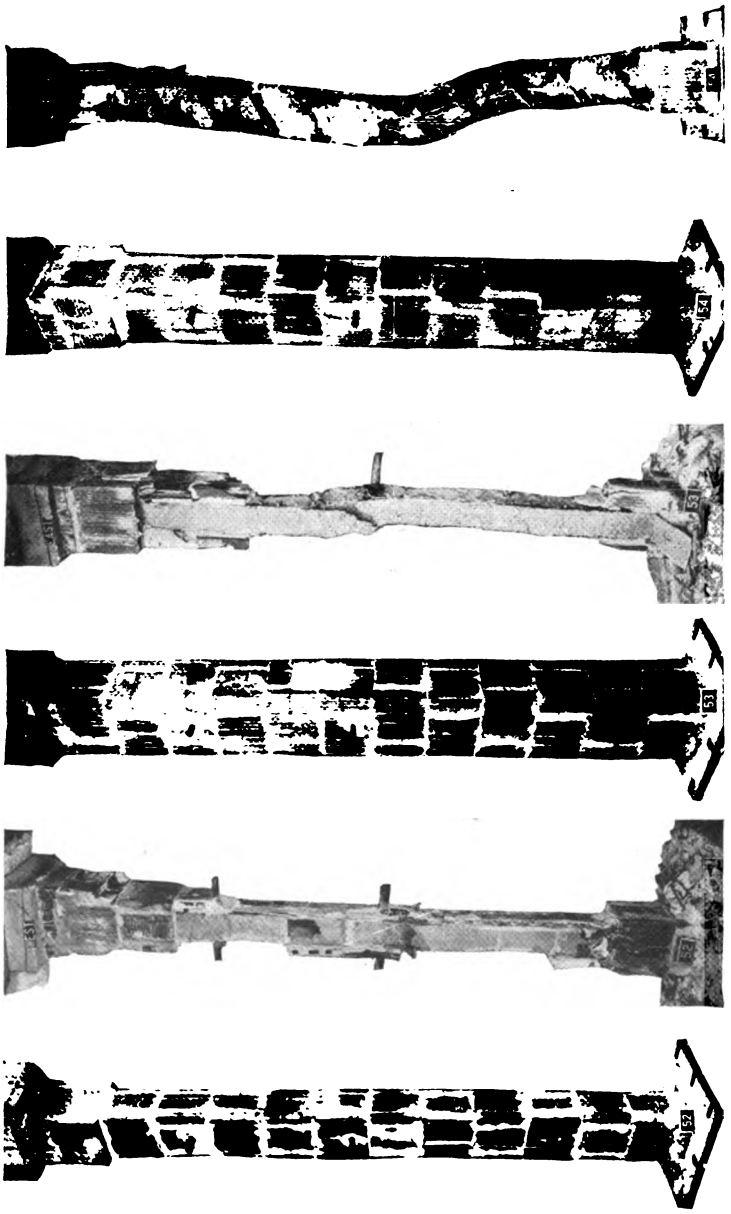

FIG. 70.—*Clay-tile protections, test Nos. 52, 53, and 54, before and after fire test*

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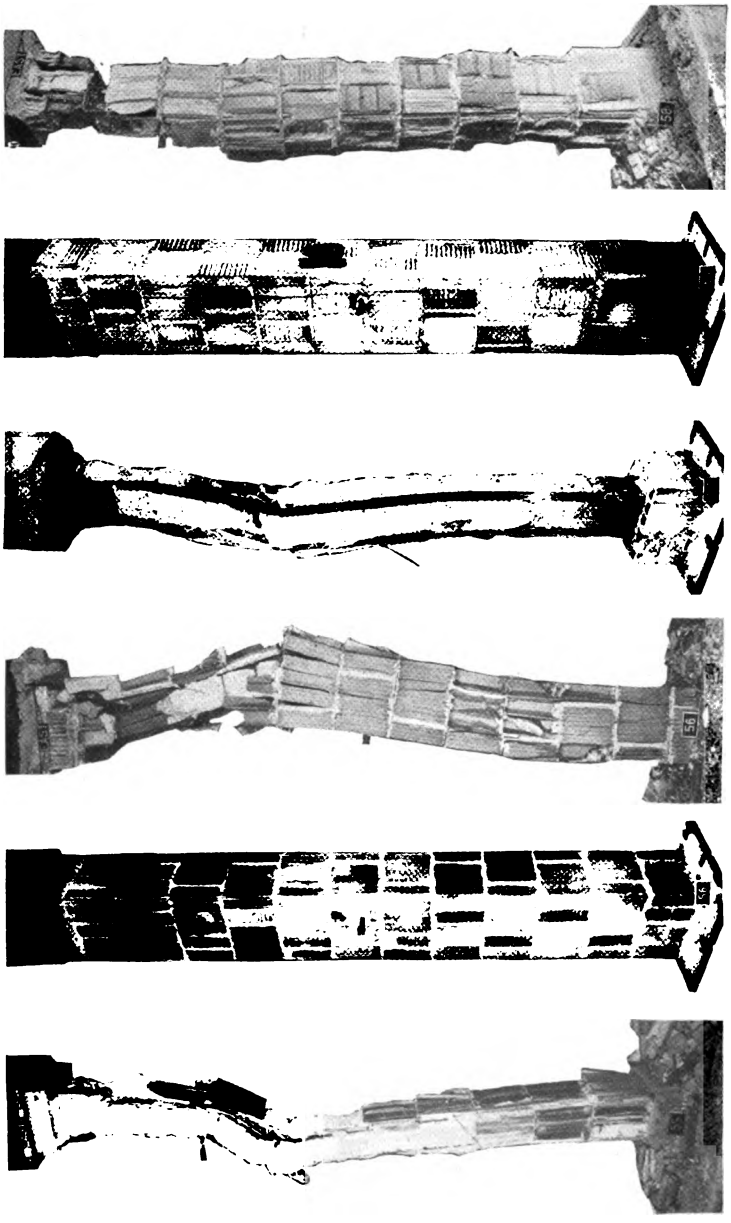

FIG. 71.—*Clay-tile protections, test Nos. 55 to 58, before and after fire test*

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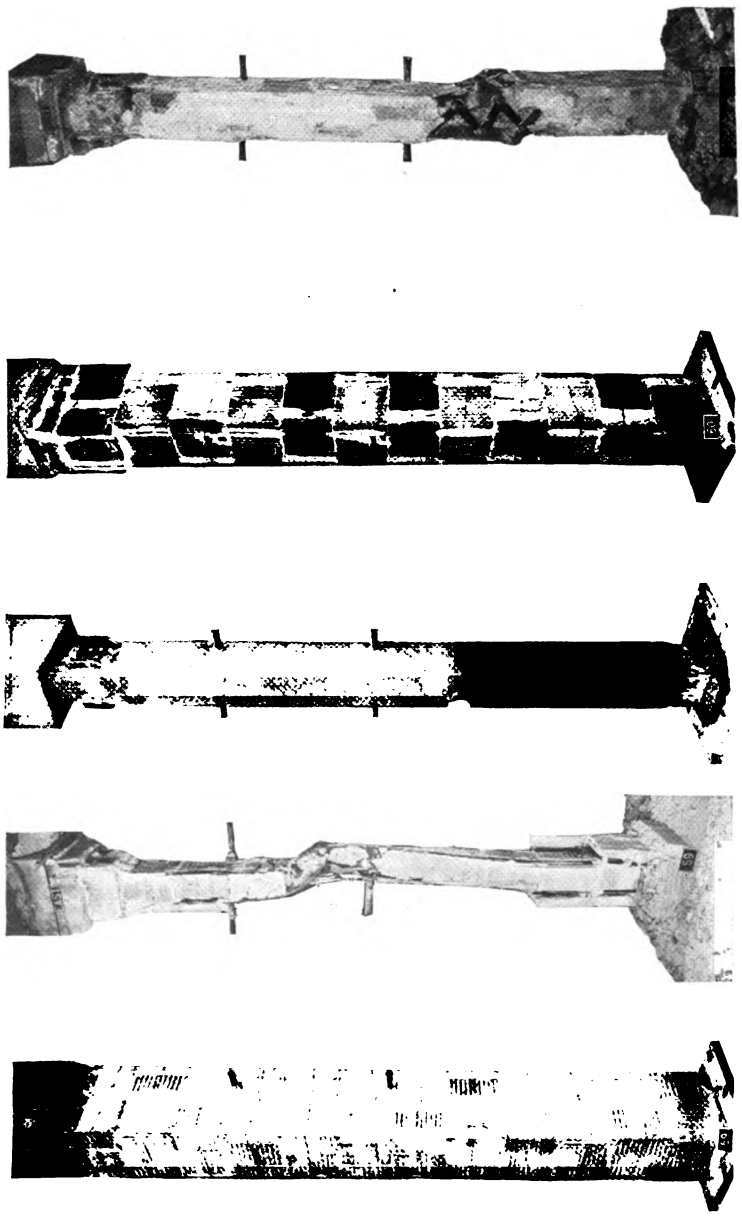

FIG. 72.—*Clay-tile protections, test Nos. 59 and 60, before and after fire test*

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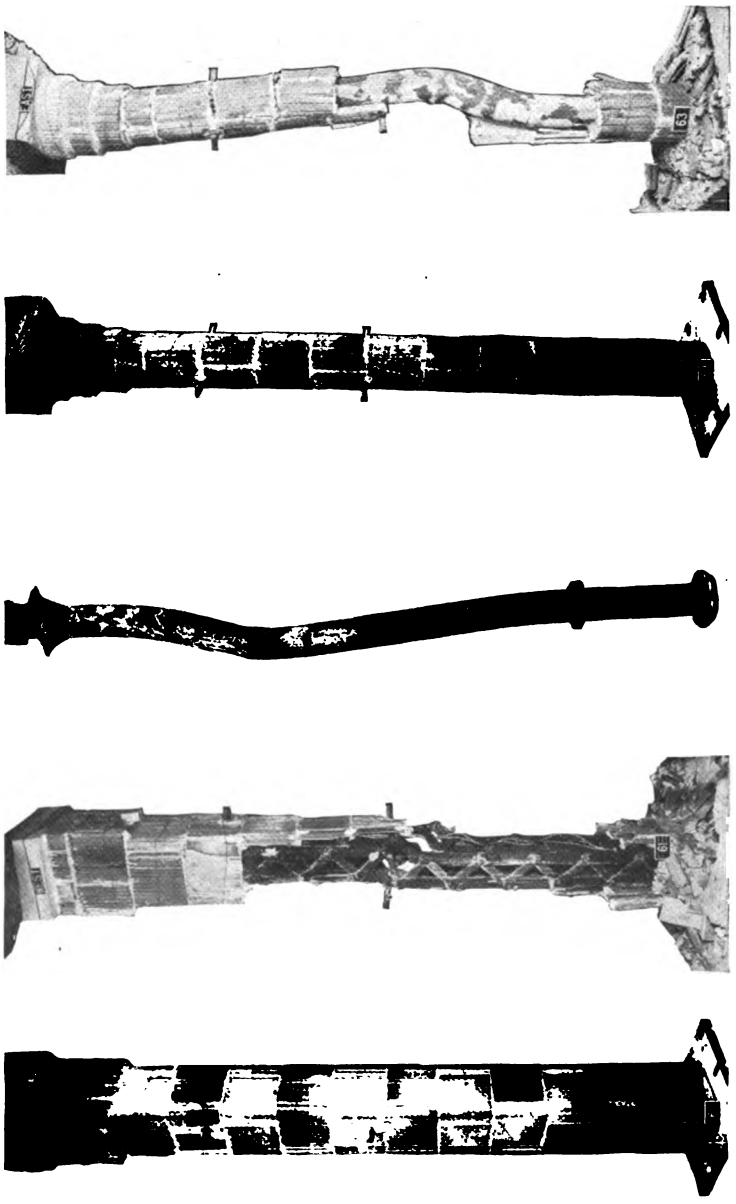

FIG. 73.—*Clay-tile projections, test Nos. 61, 62, and 63, before and after fire test*

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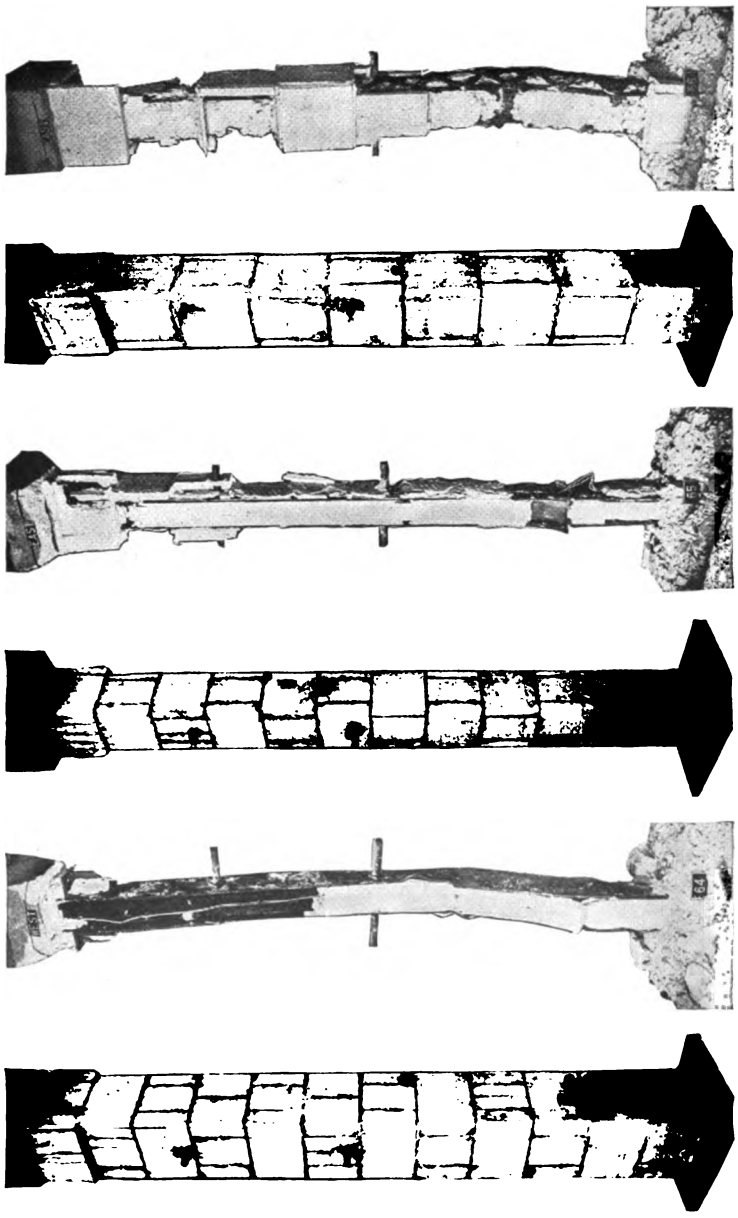

FIG. 74.—*Gypsum-block protections, test Nos. 64, 65, and 66, before and after fire test*

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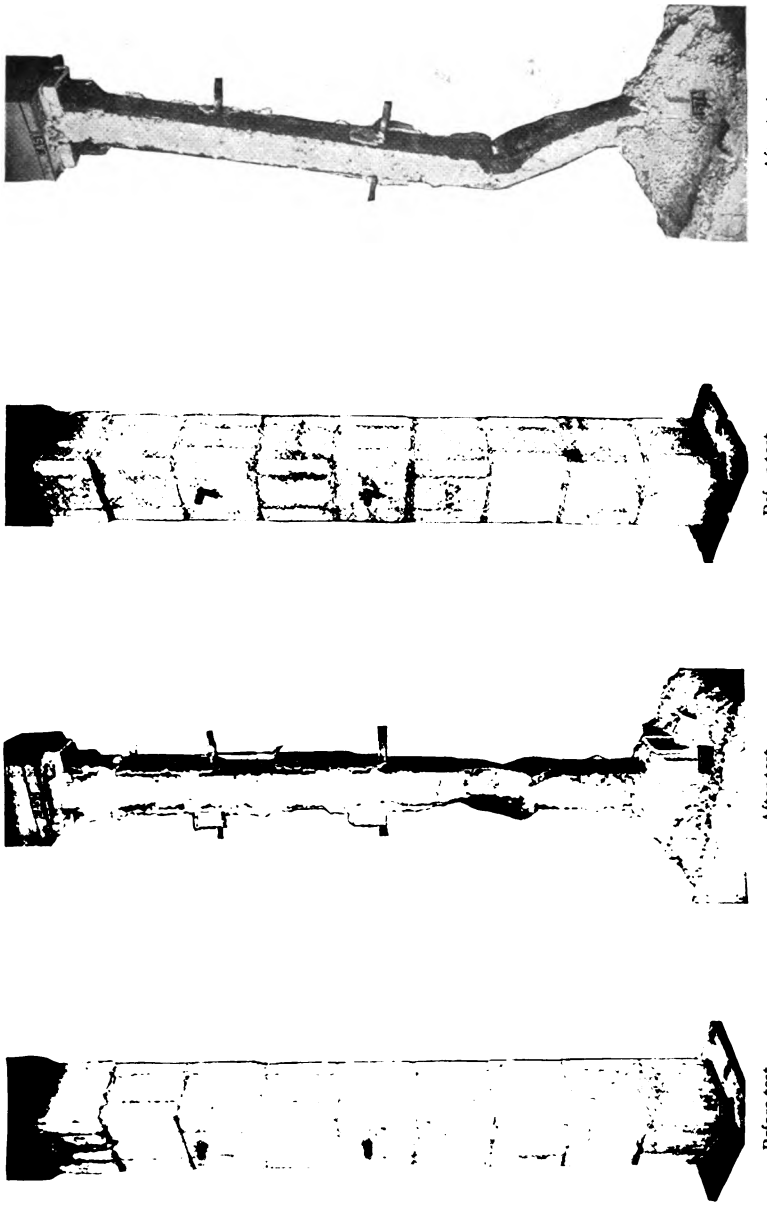

FIG. 75.—*Gypsum-block projections, test Nos. 67 and 67A, before and after fire test*

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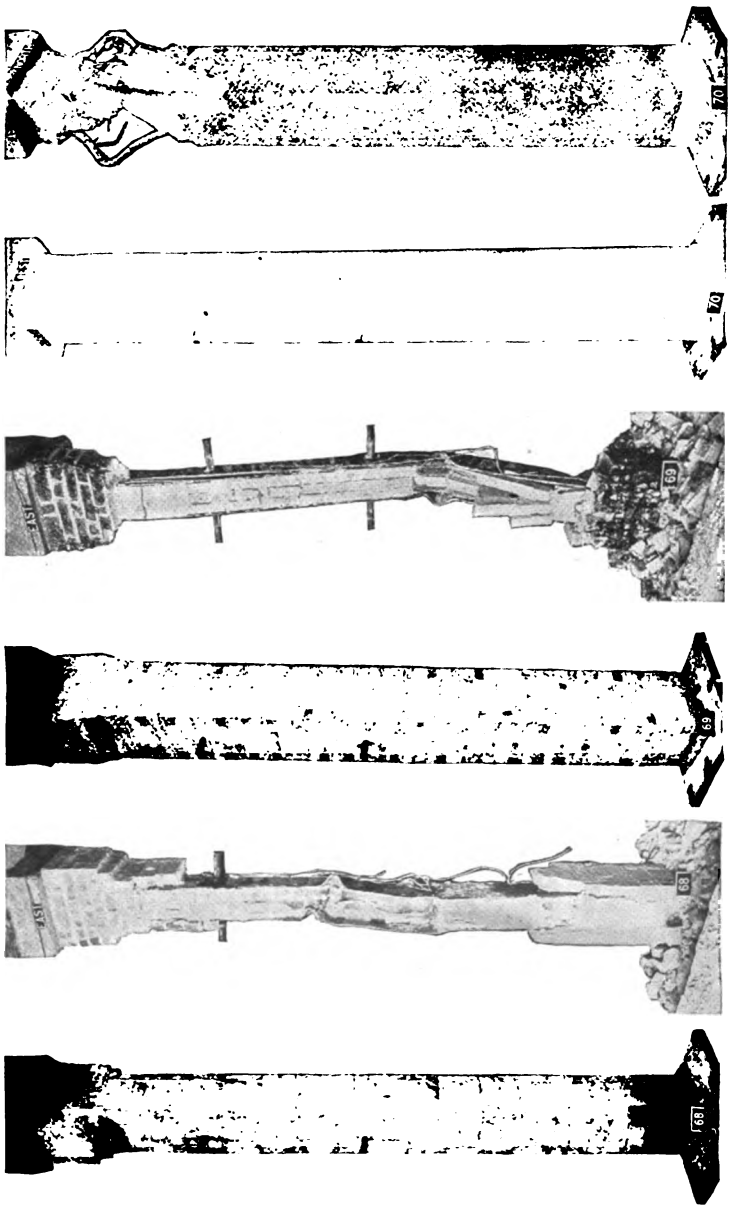

FIG. 76.—*Brick protections, test Nos. 68 and 69; reinforced-concrete column, test No. 70, before and after fire test*

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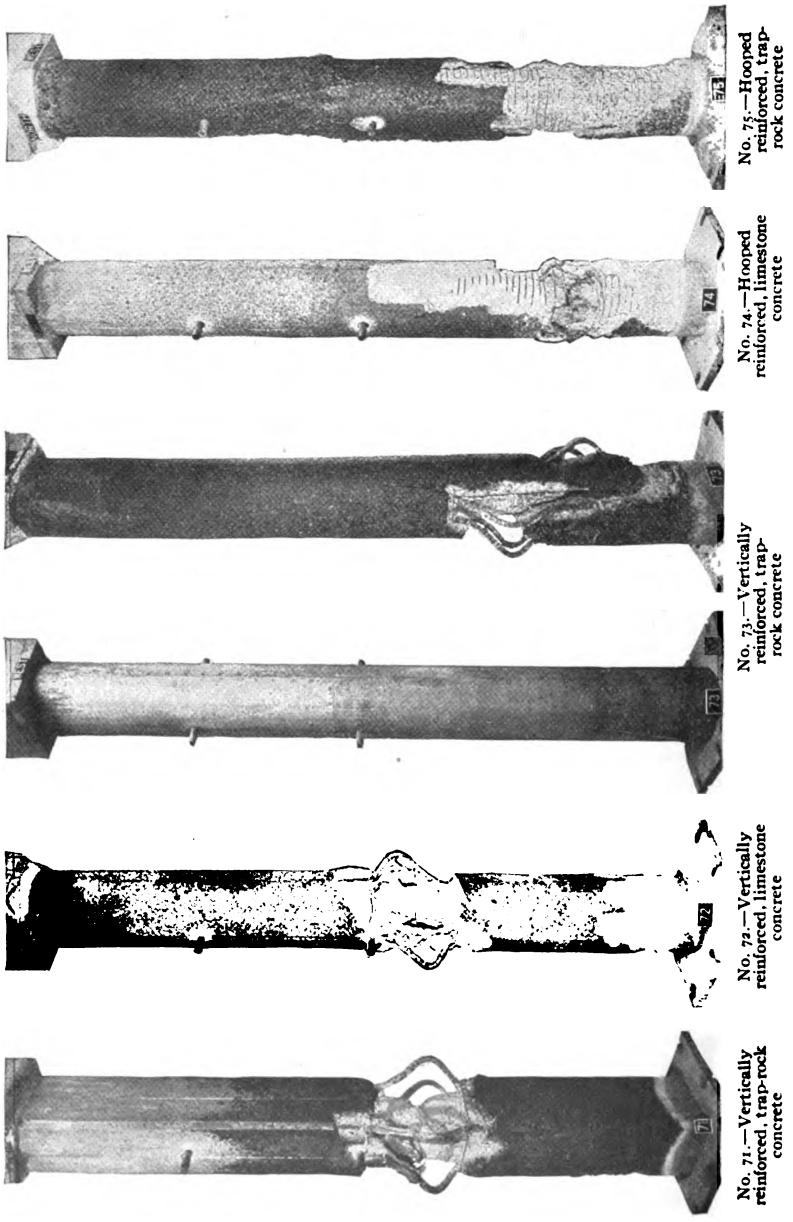
No. 71.—Vertically reinforced, trap-rock concrete
No. 72.—Vertically reinforced, limestone concrete
No. 73.—Vertically reinforced, trap-rock concrete
No. 74.—Hooped reinforced, limestone concrete
No. 75.—Hooped reinforced, trap-rock concrete

FIG. 77.—*Reinforced-concrete columns, test Nos. 71 to 75, after fire test*

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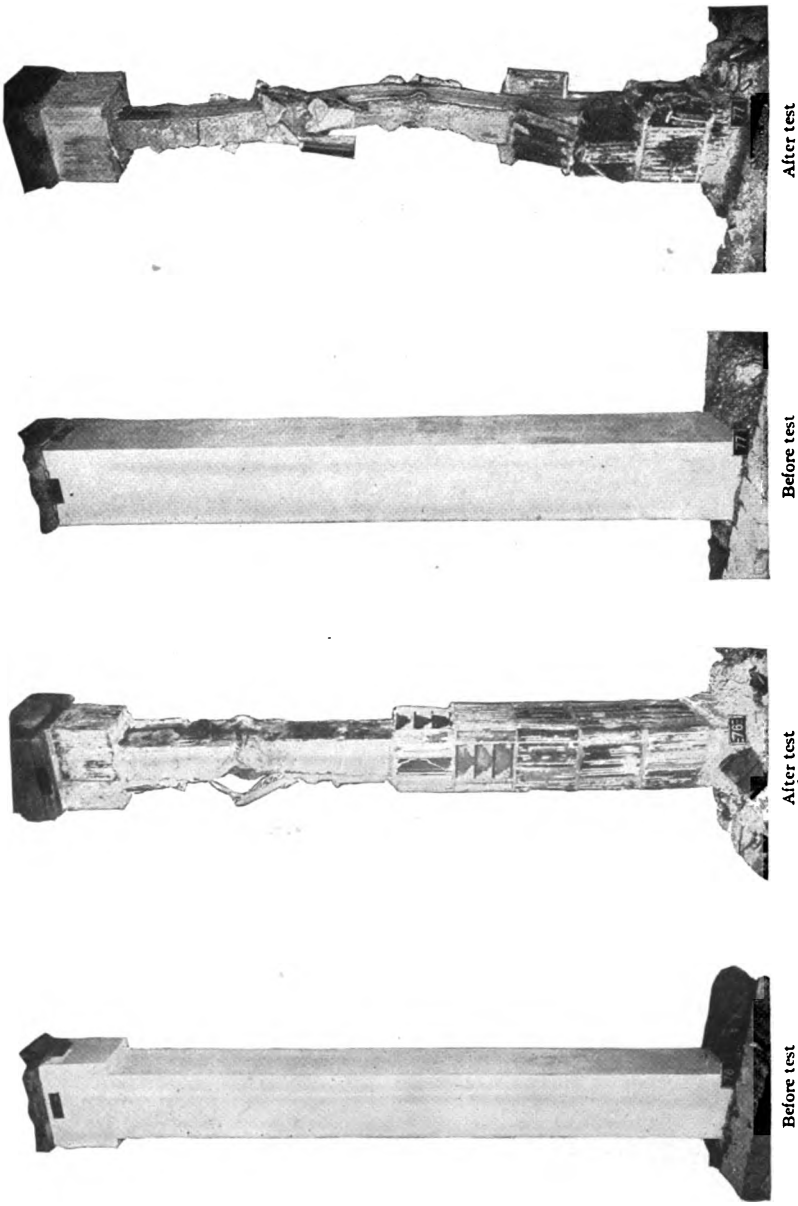

FIG. 78.—*Plastered clay-tile protections, test Nos. 76 and 77, before and after fire test*

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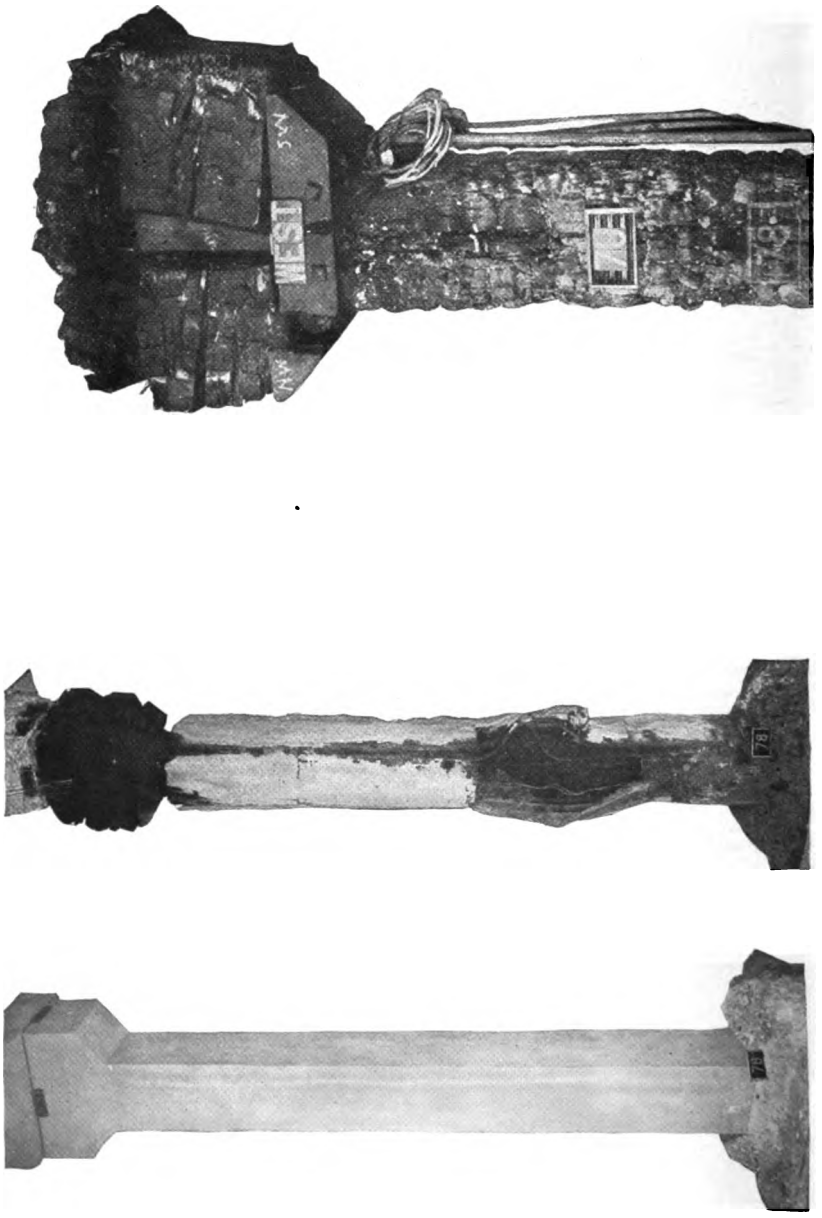

FIG. 79.—*Longleaf pine column, protected, cap and pintle bearing, test No. 78, before and after fire test*

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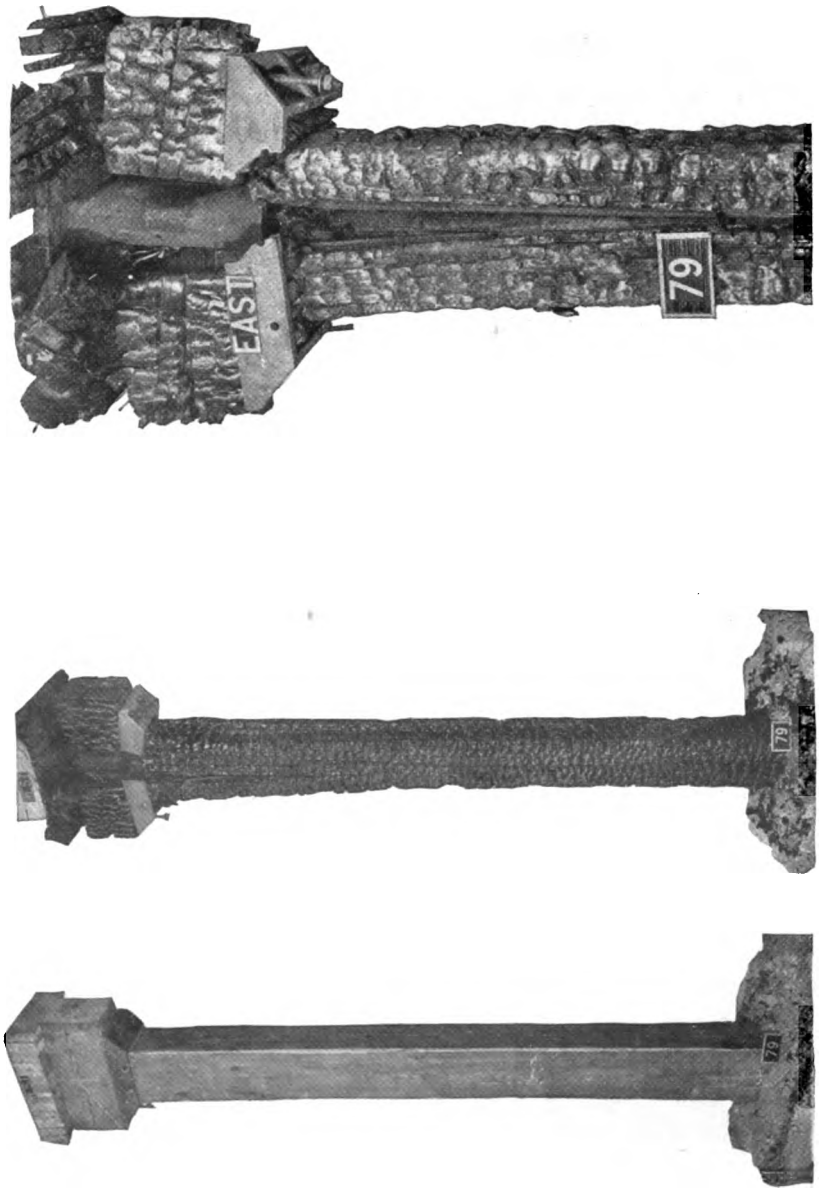

FIG. 80.—*Longleaf pine column, unprotected, cap and pintle bearing, test No. 79, before and after fire test*

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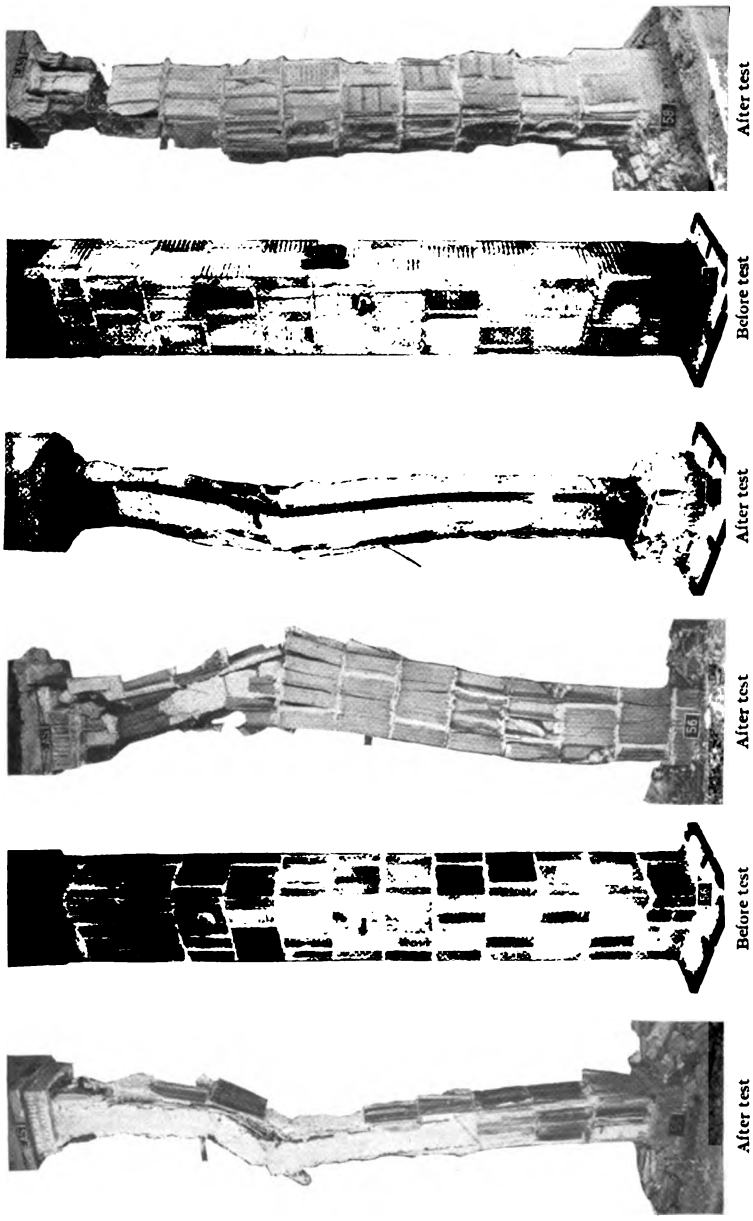

FIG. 71.—*Clay-tile protections, test Nos. 55 to 58, before and after fire test*

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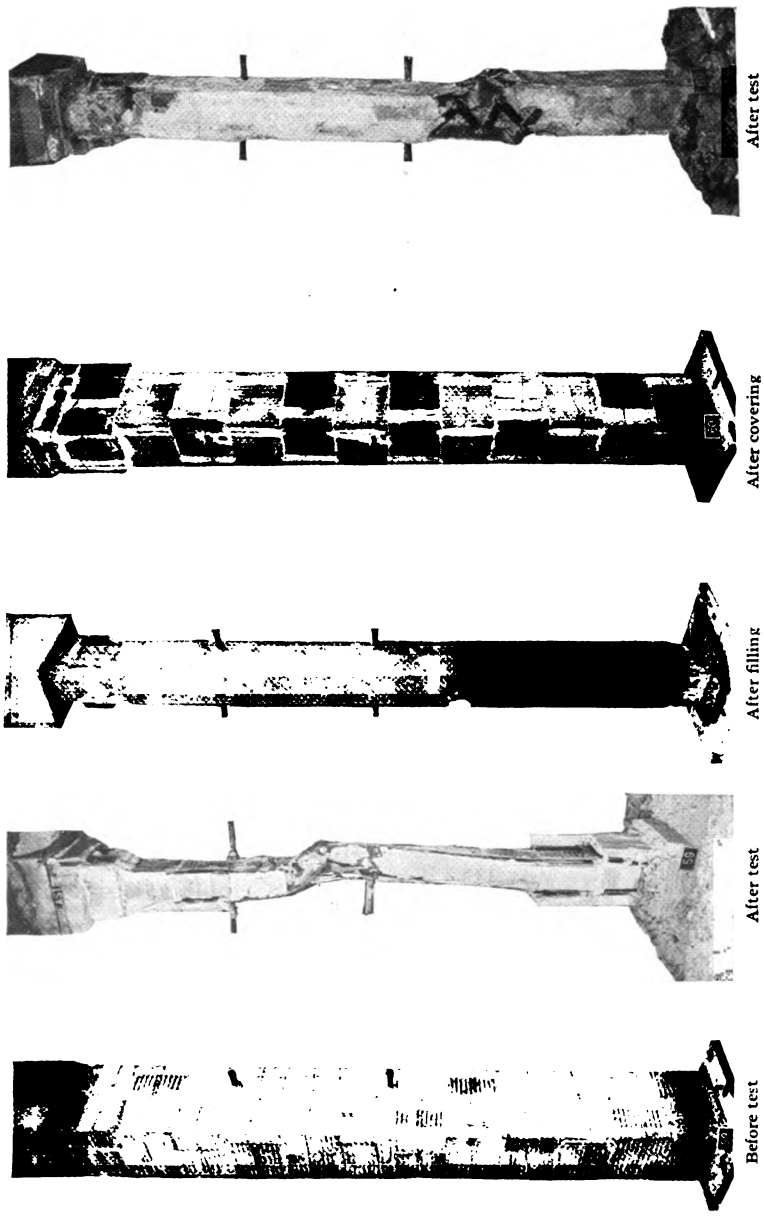

FIG. 72.—*Clay-tile protections, test Nos. 59 and 60, before and after fire test*

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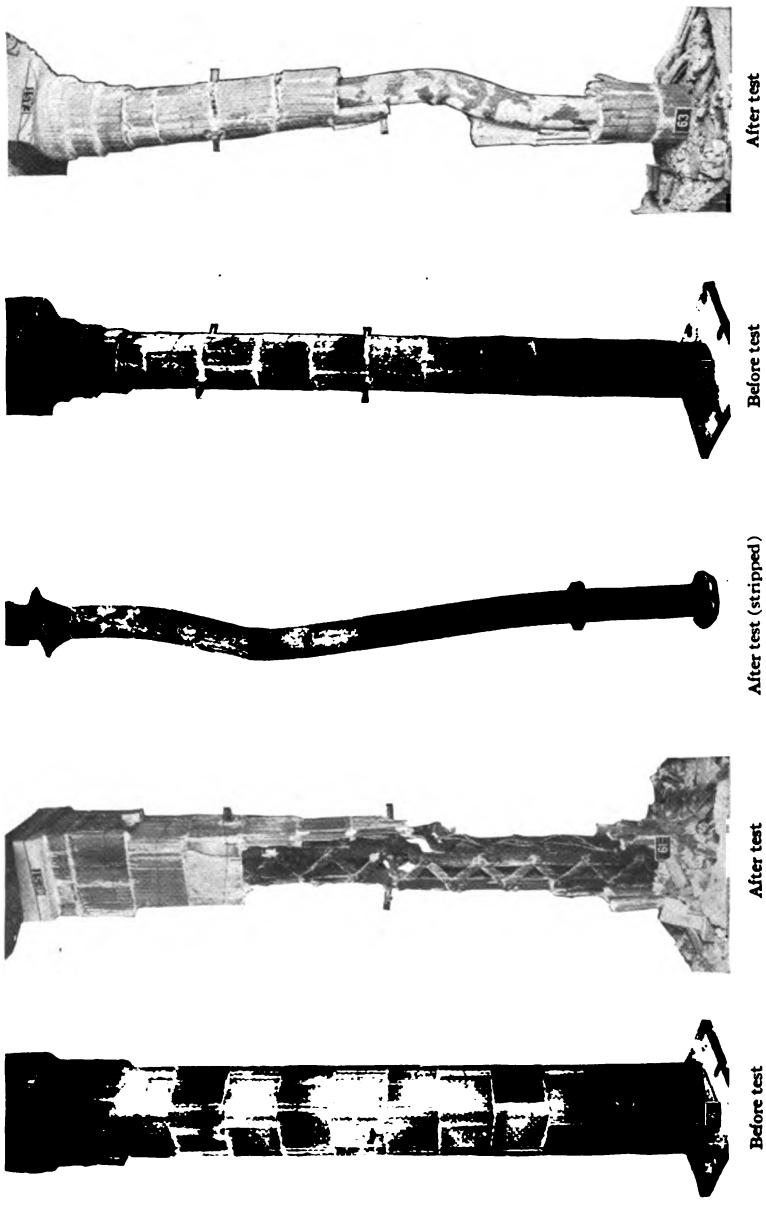

FIG. 73.—*Clay-tile protections, test Nos. 61, 62, and 63, before and after fire test*

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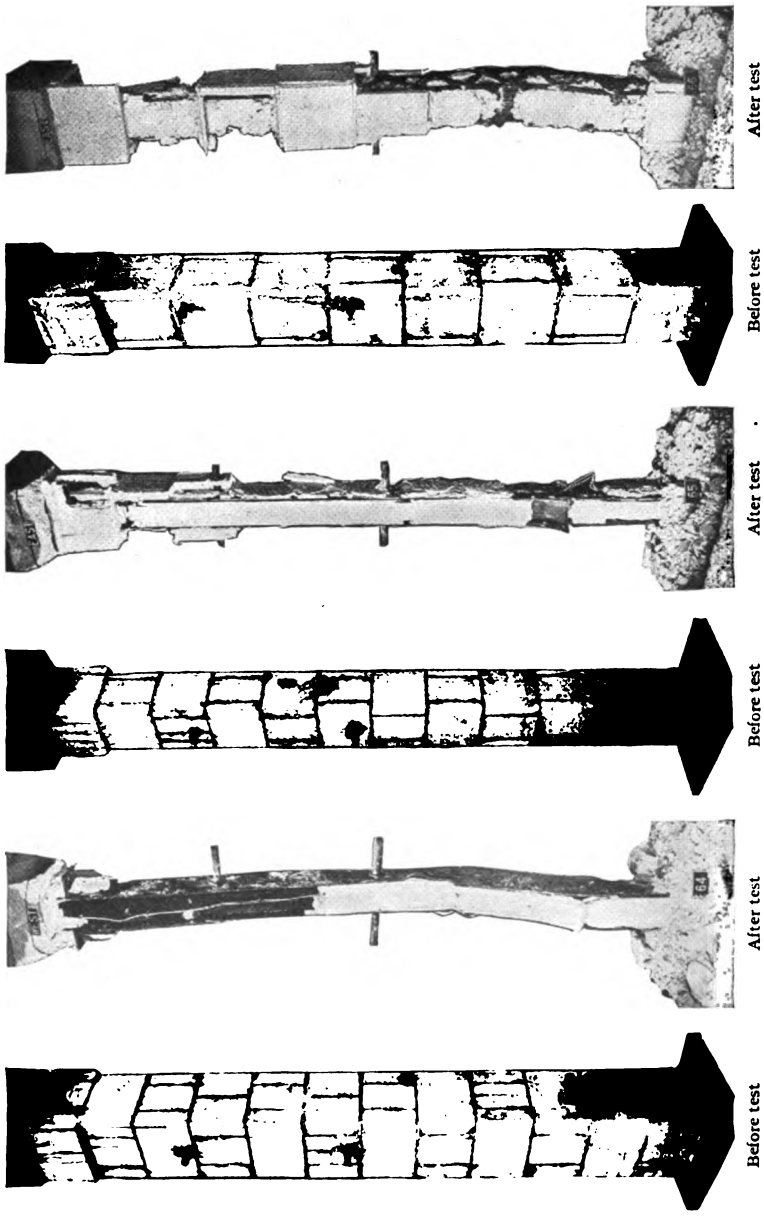

FIG. 74.—*Gypsum-block protections, test Nos. 64, 65, and 66, before and after fire test*

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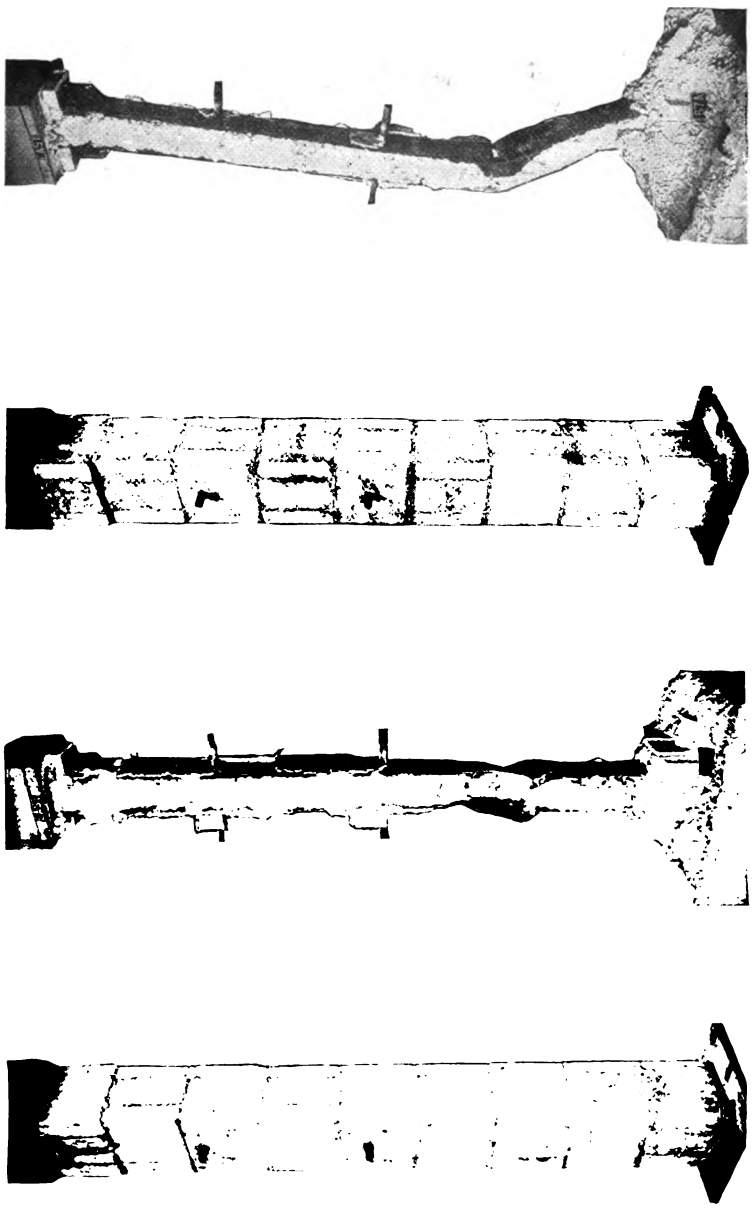

FIG. 75.—*Gypsum-block protections, test Nos. 67 and 67A, before and after fire test*

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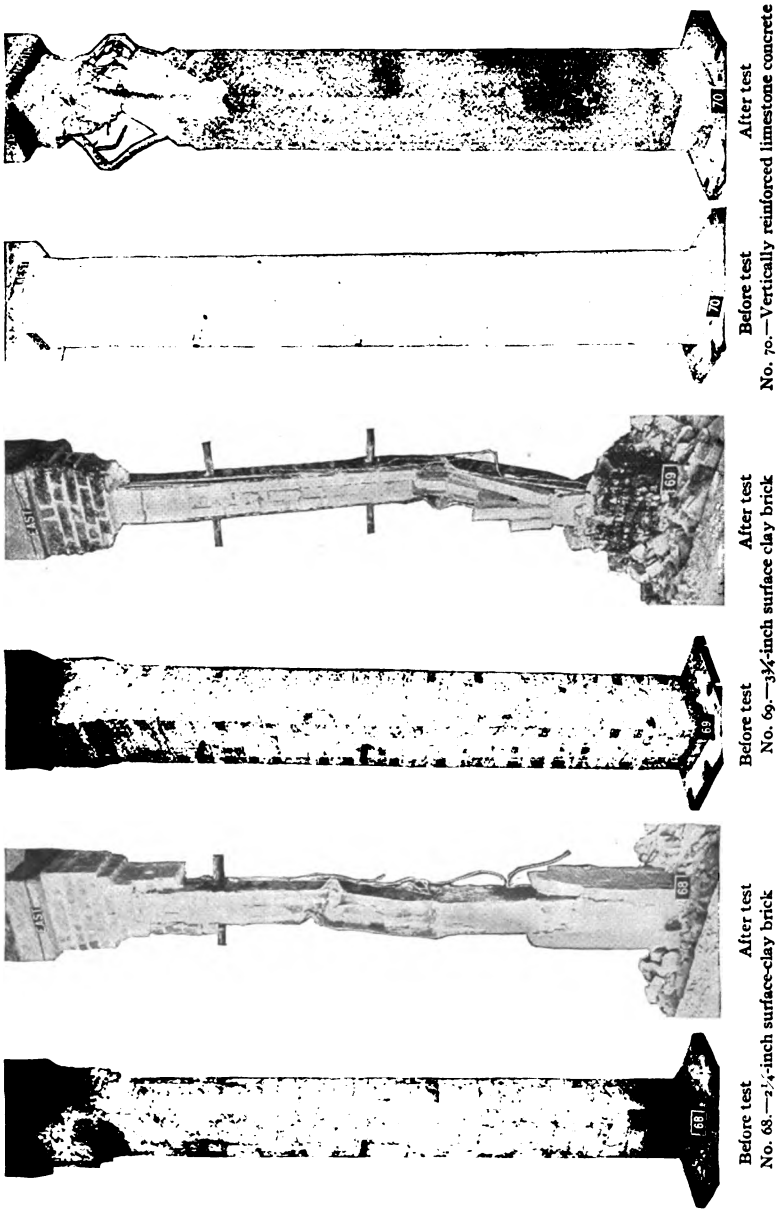

FIG. 76.—*Brick protections, test Nos. 68 and 69; reinforced-concrete column, test No. 70, before and after fire test*

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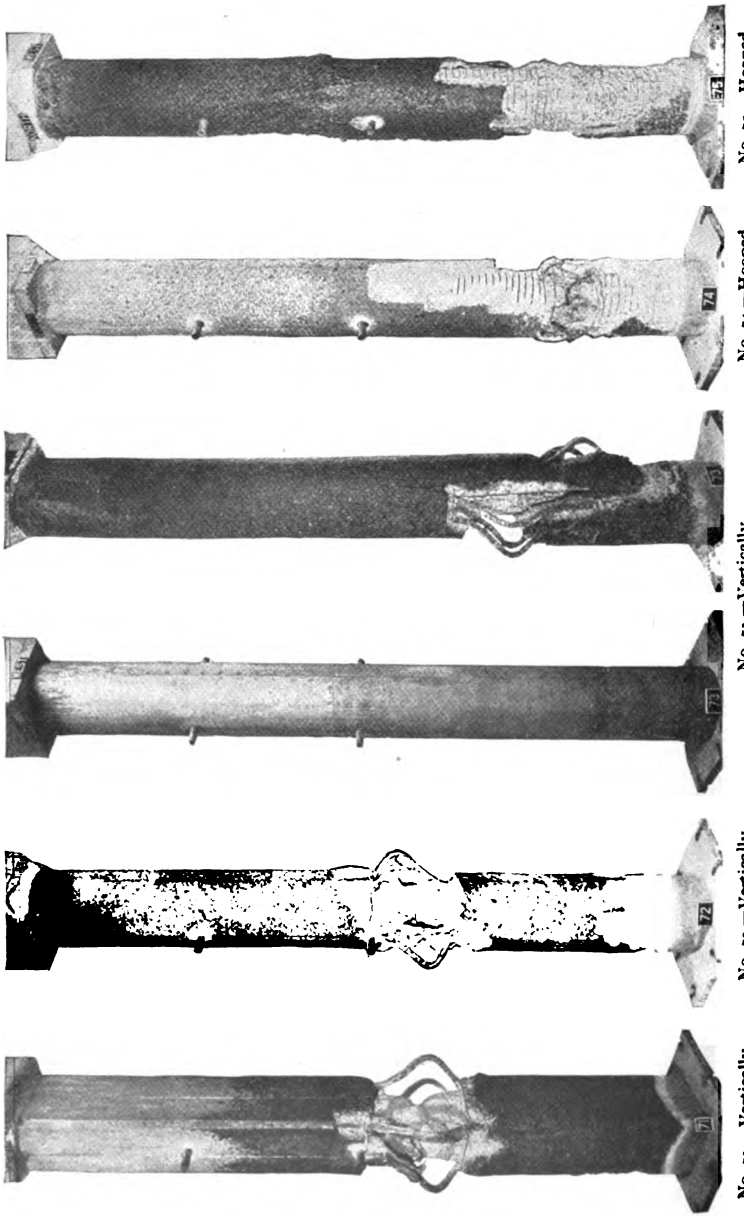
No. 71.—Vertically reinforced, trap-rock concrete
No. 72.—Vertically reinforced, limestone concrete
No. 73.—Vertically reinforced, trap-rock concrete
No. 74.—Hooped reinforced, limestone concrete
No. 75.—Hooped reinforced, trap-rock concrete

FIG. 77.—*Reinforced-concrete columns, test Nos. 71 to 75, after fire test*

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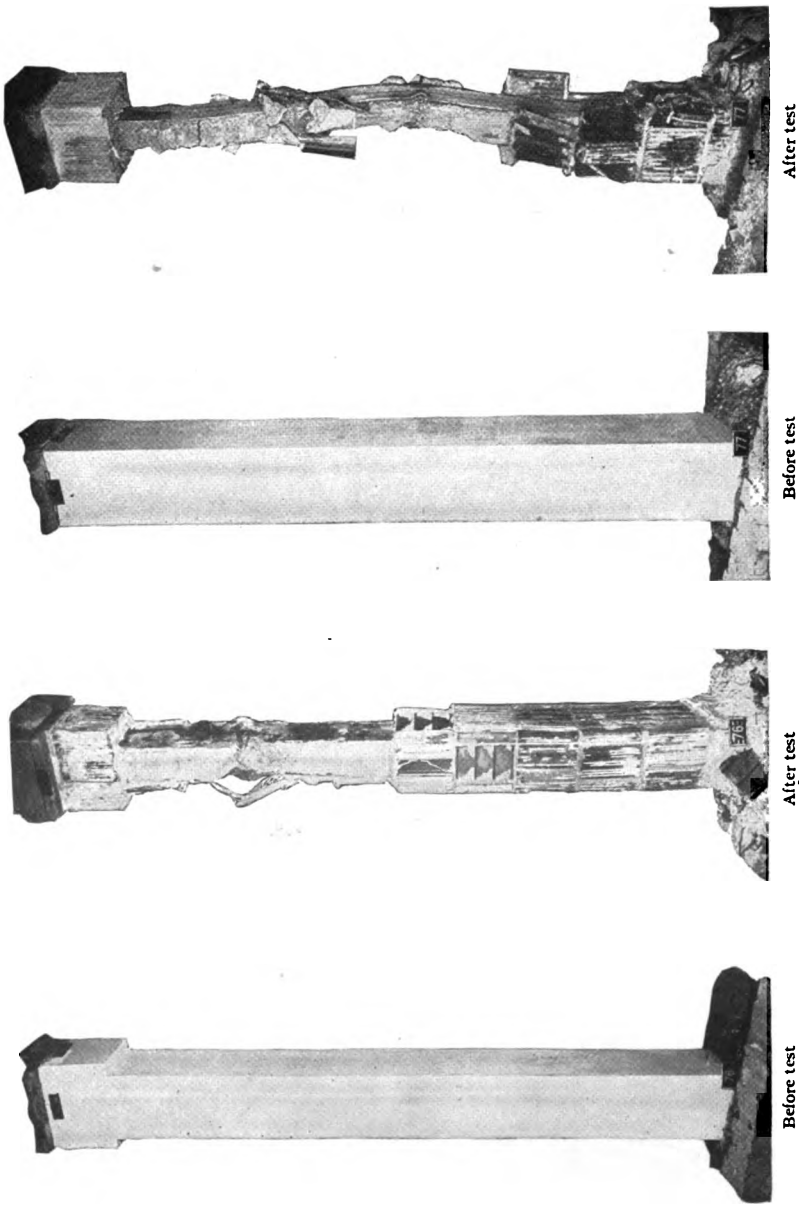

FIG. 78.—*Plastered clay-tile protections, test Nos. 76 and 77, before and after fire test*

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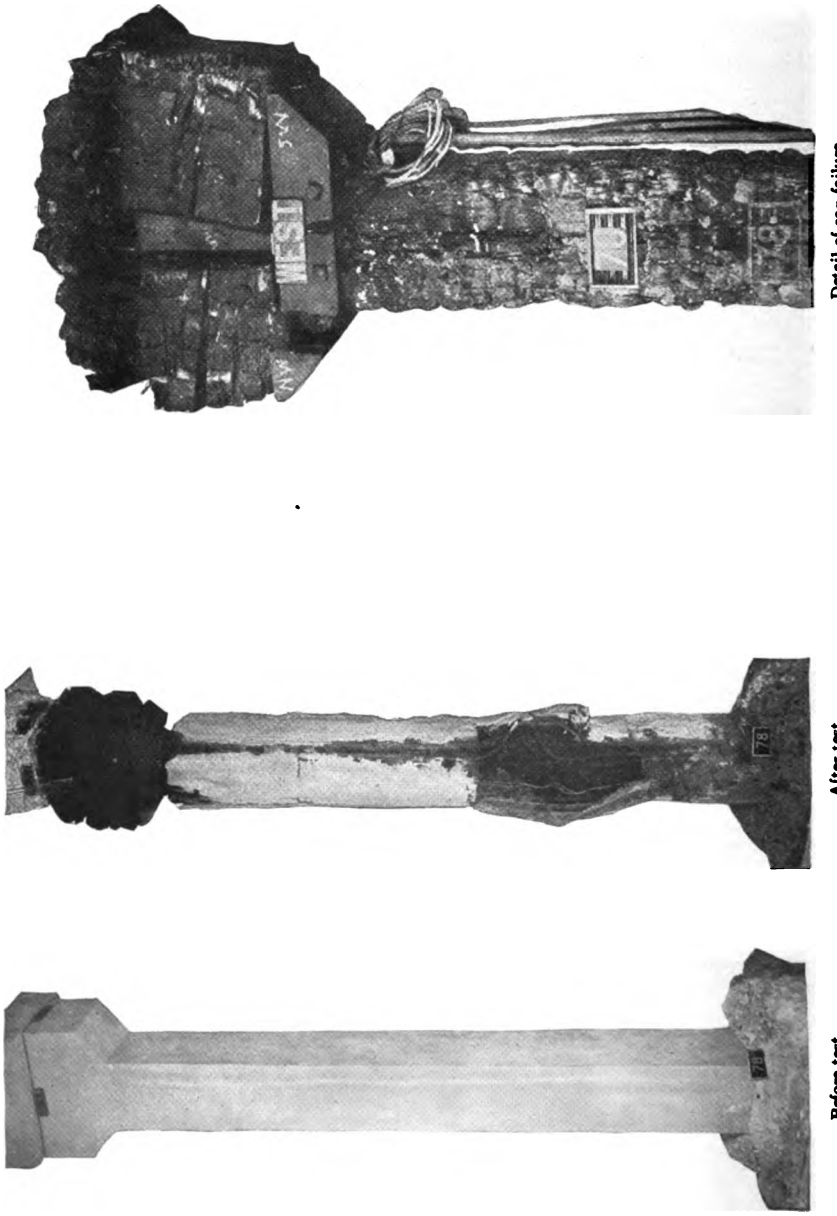

Detail of cap failure

After test

Before test

FIG. 79.—*Longleaf pine column, protected, cap and pintle bearing, test No. 78, before and after fire test*

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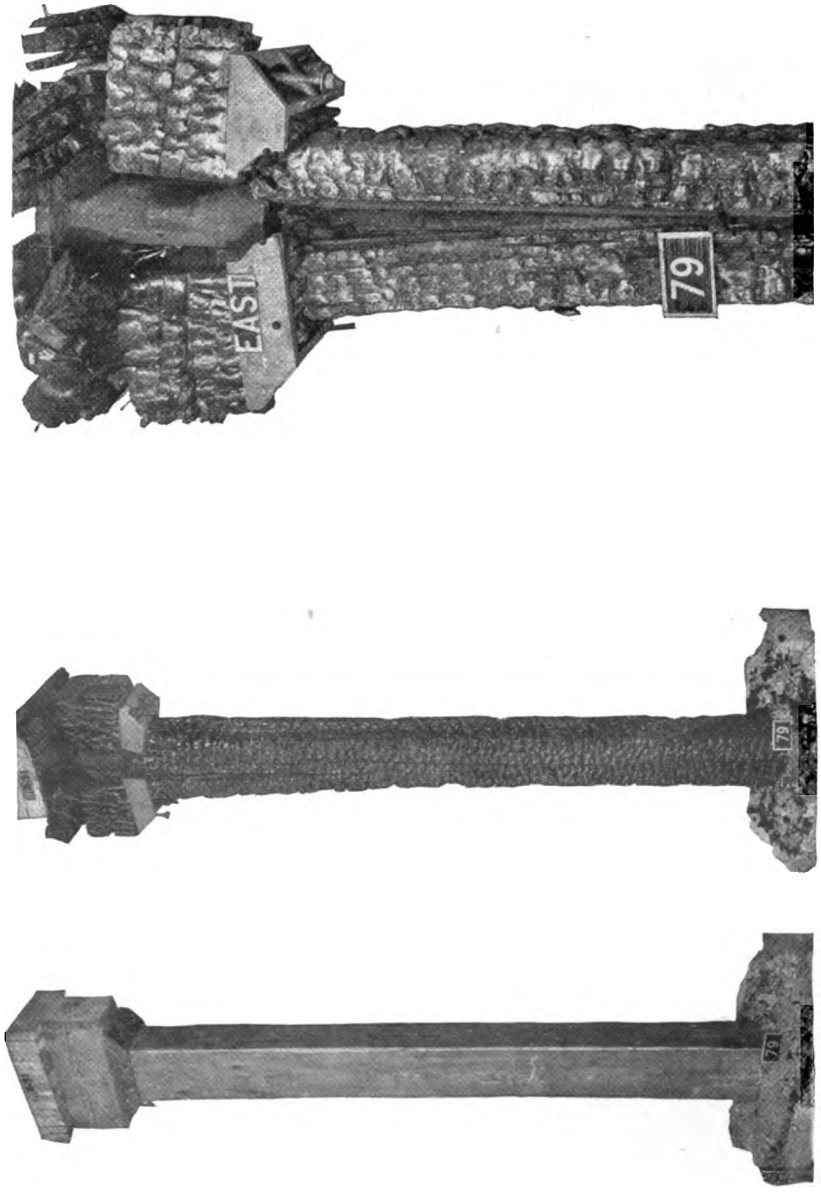

FIG. 80.—*Longleaf pine column, unprotected, cap and pintle bearing, test No. 79, before and after fire test*

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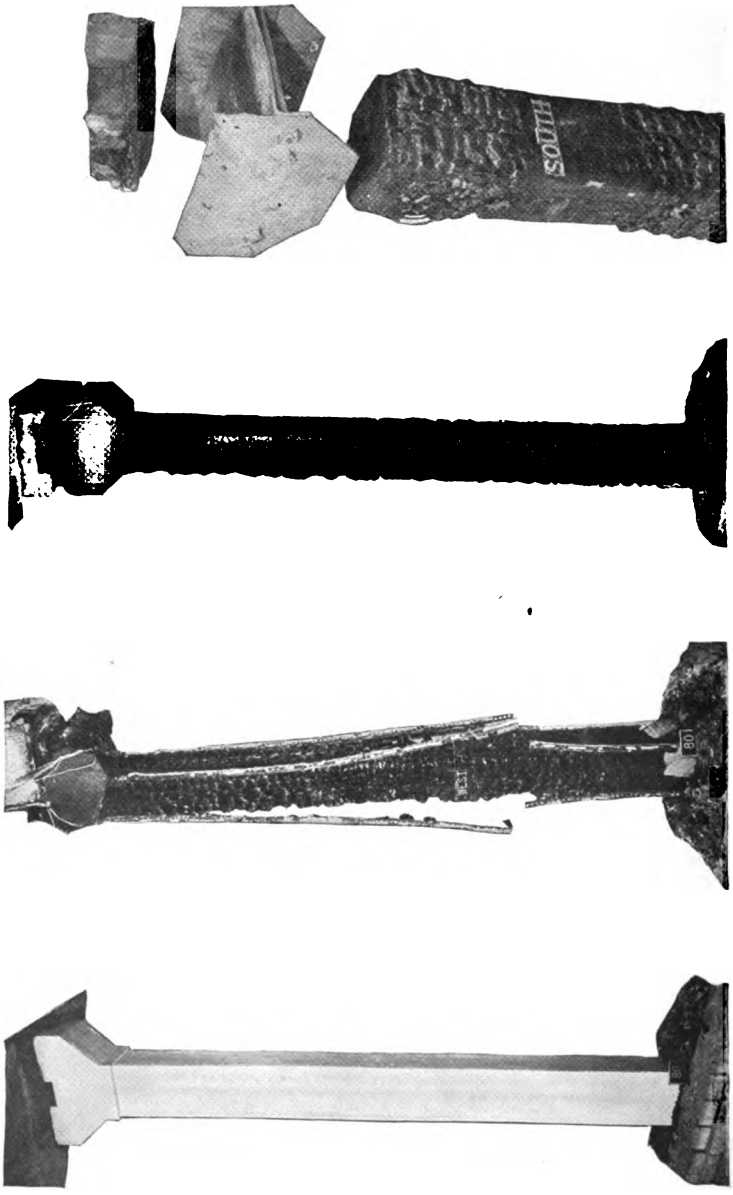

FIG 81.—*Longleaf pine columns, steel-plate cap bearings, test Nos. 80 and 81, before and after fire test*

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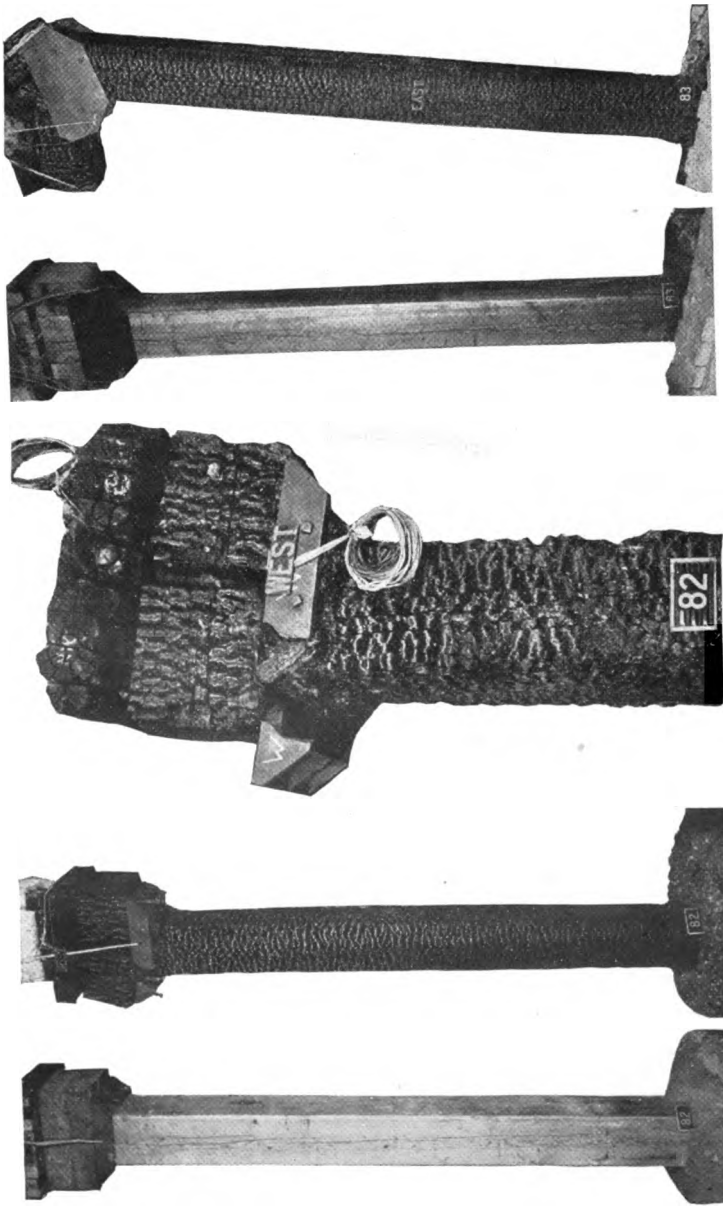

FIG. 82.—*Douglas fir columns, unprotected, test Nos. 82 and 83, before and after fire test*

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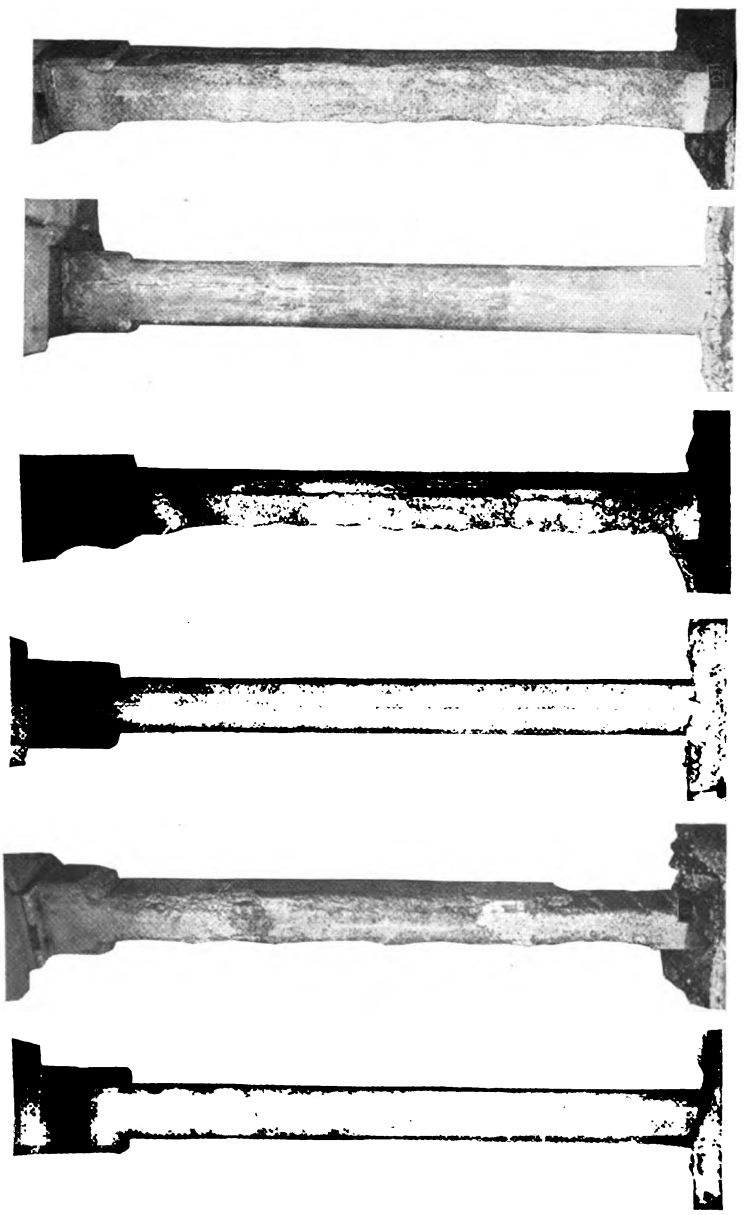

FIG. 83.—*Concrete protections, test Nos. 101 to 103, after fire and water tests*

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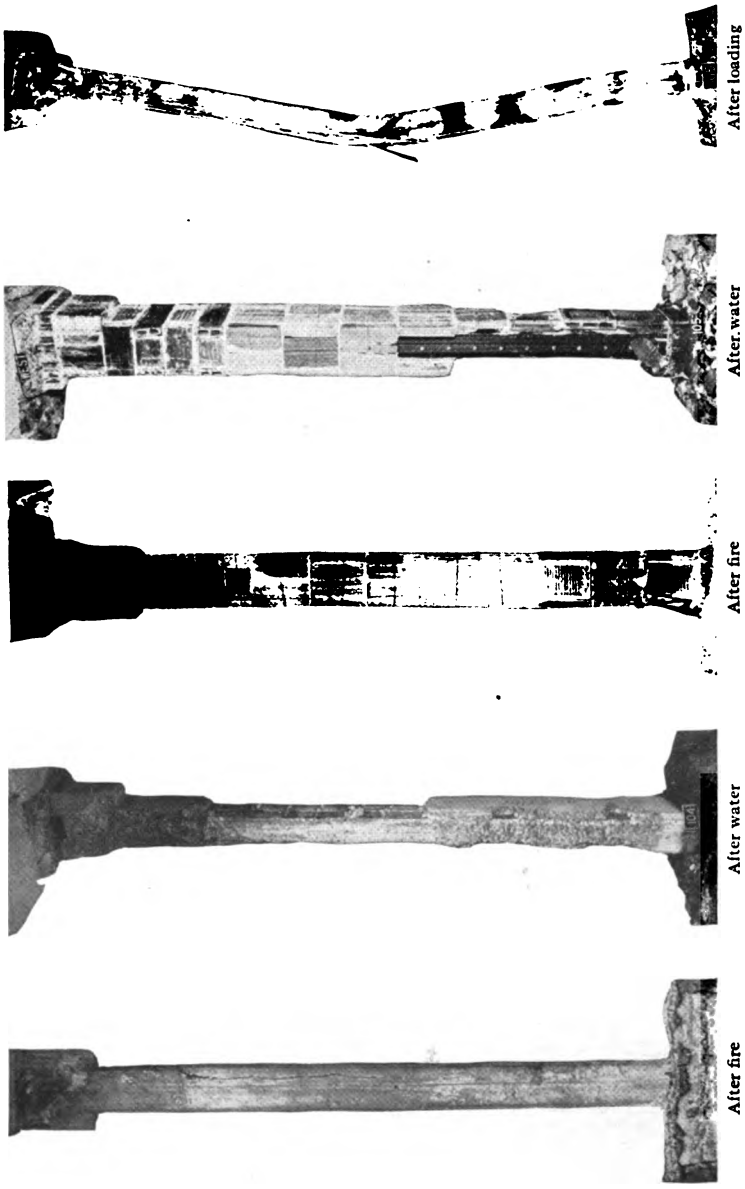

FIG. 84.—*Concrete protection, test No. 104; clay-tile protection, test No. 105, after fire and water tests*

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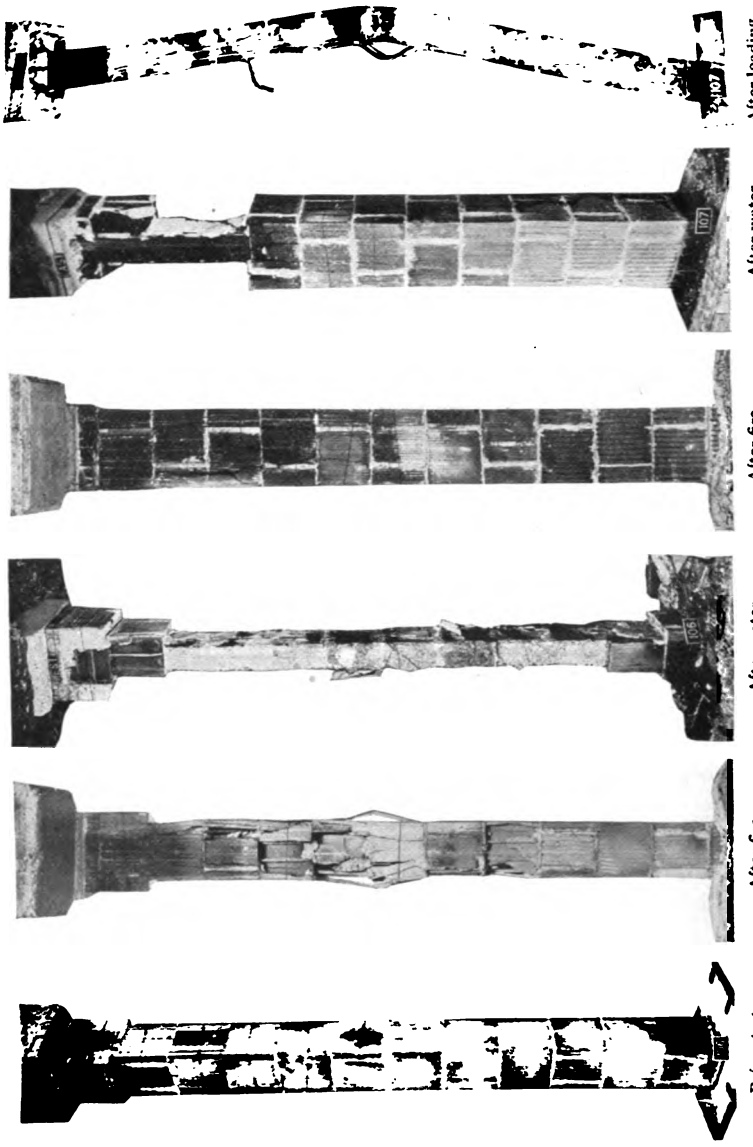

FIG. 85.—*Clay-tile projections, test Nos. 106 and 107, after fire and water tests*

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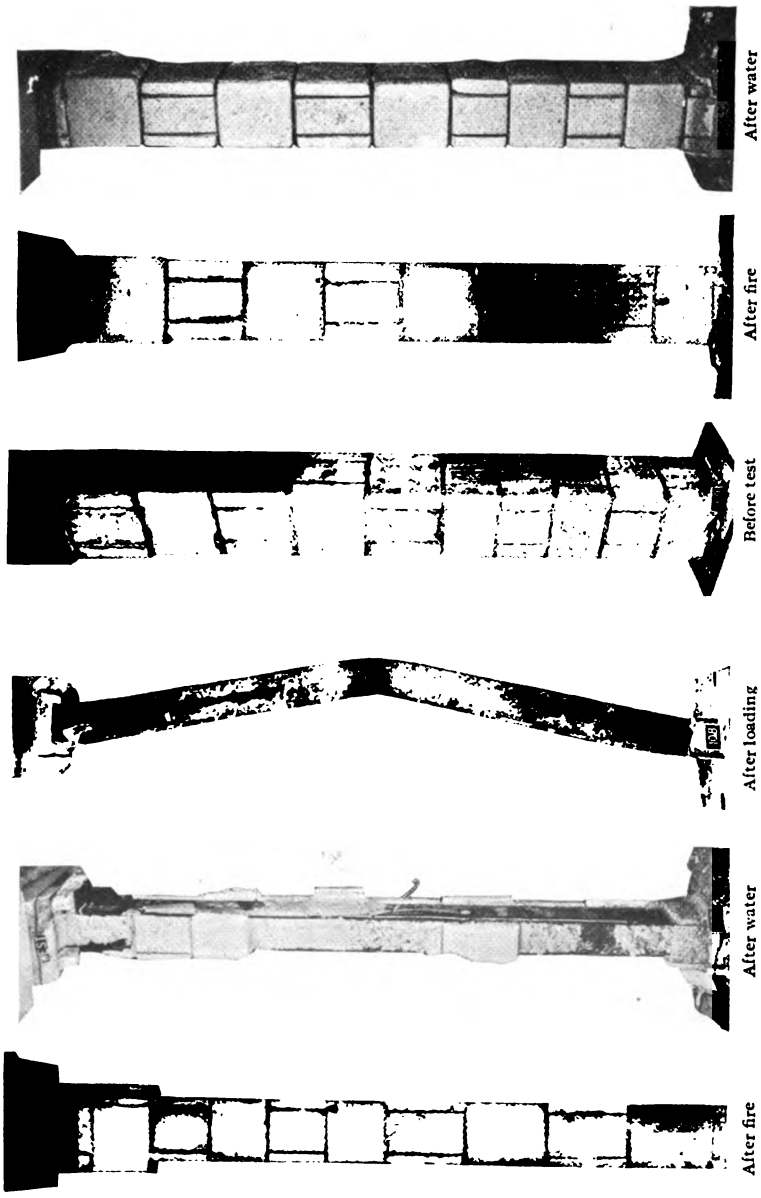

FIG. 86.—*Gypsum-block protections, test Nos. 108 and 109, after fire and water tests*

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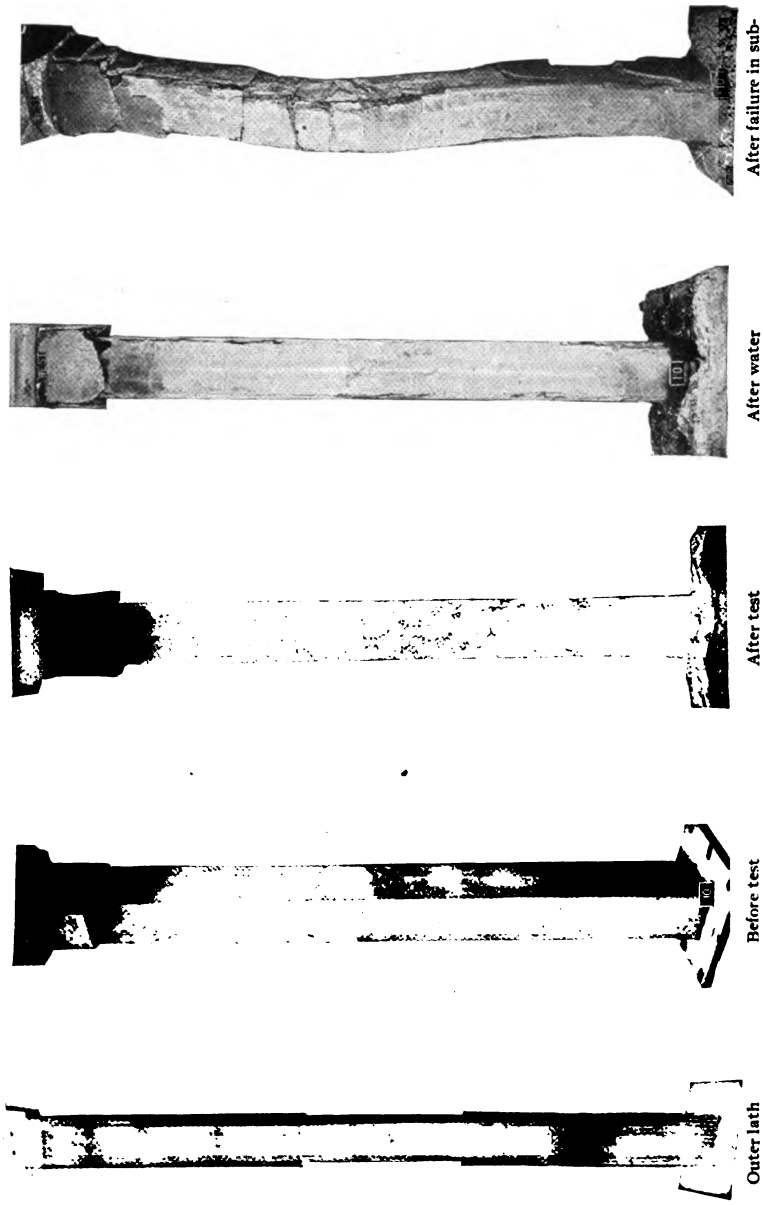

FIG. 87.—*Plaster on metal-lath protection, test No. 110, before and after fire and water tests*

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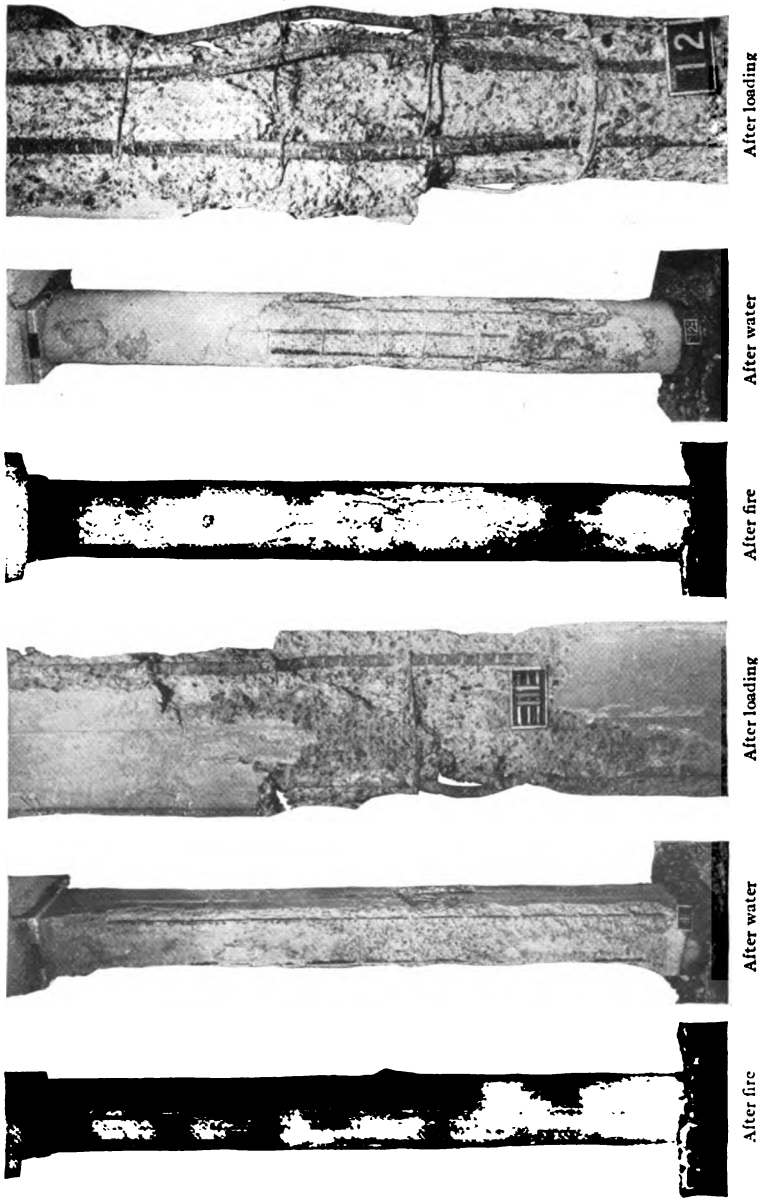

FIG. 88.—*Reinforced-concrete columns, tests Nos. 111 and 112, after fire and water tests*

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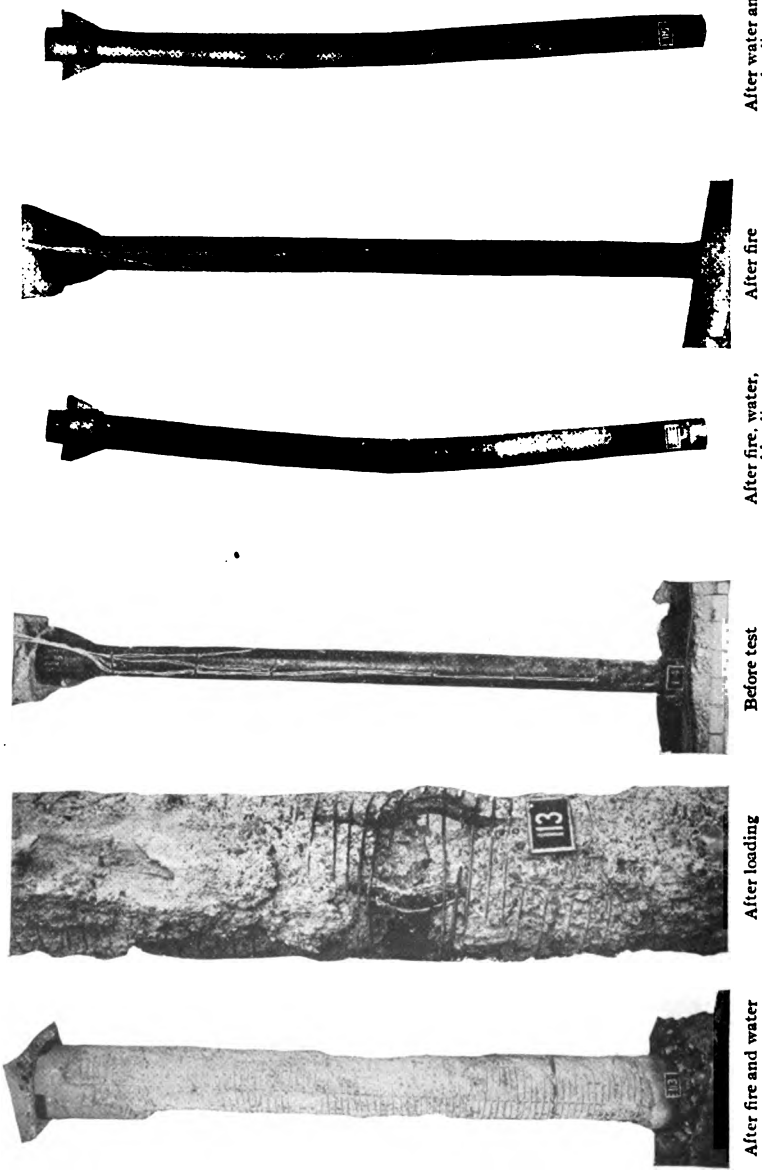

FIG. 89.—*Reinforced-concrete column, test No. 113; cast-iron columns, test Nos. 114 and 115, after fire and water tests*

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APPENDIX B.—Time-Temperature Curves

Fig. No.	Test No.	Columns and protections	Page
90	1 to 4	Unprotected structural steel	251
91	5 to 8	....do	252
92	9 to 11	Unprotected cast iron	253
93	12, 13	Unprotected pipe	254
94	14 to 17	Partly protected structural steel	255
95	18, 22	....do	256
96	19, 20, 21	....do	257
97	23, 24	Double layer of plaster on metal lath	258
90	25, 26, 27	Single layer of plaster on metal lath	259
99	28	2-inch limestone concrete	260
100	28A	....do	261
101	29	2-inch trap-rock concrete	262
102	30	2-inch Joliet gravel concrete	263
103	31	2-inch sandstone concrete	264
104	32, 32A	2-inch cinder concrete	265
105	33	4-inch limestone concrete	266
106	33A	....do	267
107	34	4-inch granite concrete	268
108	34A	....do	269
109	35	4-inch limestone concrete	270
103	36	2-inch trap-rock concrete	264
110	37	4-inch trap-rock concrete	271
111	38	2-inch Joliet gravel concrete	272
112	39	4-inch Meramec River gravel concrete	273
113	40	2-inch trap-rock concrete	274
114	41	4-inch limestone concrete	275
115	42	....do	276
116	43, 44	2-inch sandstone concrete	277
112	45	2-inch Meramec River gravel concrete	273
117	46	2-inch trap-rock concrete (3½ inch outside of angles)	278
101	47	2-inch cinder concrete	262
118	48, 49	Semifire-clay tile	279
119	50, 50A	Surface-clay tile	280
120	51, 51A	....do	281
121	52, 53	Shale tile	282
122	54, 55	Semifire-clay tile	283
123	56	Semifire clay	284
123	57	Surface-clay tile	284
124	58, 59	....do	285
125	60, 61	Semifire-clay tile	286
126	62, 63	Porous semifire-clay tile	287
127	64	4-inch gypsum block	288
128	65, 66	2-inch gypsum block	289
129	67	4-inch gypsum block	290
130	67A	....do	291
118	68	2¼-inch surface-clay brick	279
131	69	3¼-inch surface-clay brick	292
132	70	Square, reinforced, limestone concrete	293
133	71	Square, reinforced, trap-rock concrete	294
134	72	Round, reinforced, limestone concrete	295

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APPENDIX B.—Time-Temperature Curves—Continued

Fig. No.	Test No.	Columns and protections	Page
135	73	Round, reinforced, trap-rock concrete	296
136	74	Round, hooped, limestone concrete	297
137	75	Round, hooped, trap-rock concrete	298
138	76	Plastered-clay tile	298
139	77	....do	300
140	78, 79, 82	Timber columns, cap and pintle	301
141	80, 81, 83	Timber columns, steel-plate cap	302
142	101 to 104	Concrete, fire and water tests	303
143	105 to 107	Clay tile, fire and water tests	304
143	108, 109	Gypsum block, fire and water tests	304
144	110	Plaster on metal lath, fire and water tests	305
145	111 to 113	Reinforced concrete, fire and water tests	305
145	114, 115	Unprotected cast iron, fire and water tests	306

NOTES

The upper curves in the diagrams, usually four in number, give indicated furnace temperatures at four points within the furnace chamber, two at 3 feet and two at 9 feet above the fireproofing line at the base of the test column, the lower location being designated by *L* and the upper *U*. The other letters of the curve designations indicate the corner of the furnace nearest to the given thermocouple location. Thus:

L–NW, lower northwest location
L–SE, lower southeast location
U–NE, upper northeast location
U–SW, upper southwest location

cf. Section VII, paragraph *2a*, and Fig. 31.

The lower curves in the diagrams give temperatures at four general levels on the test column, and for one or two of the levels the temperatures are given for several points on the section, as shown by the location diagram given for each test. The levels are indicated by the letters *L*, *N*, *M*, and *T*, for locations 1½, 4½, 7½, and 10½ feet, respectively, above the fireproofing line at the base of the column. Thus for test No. 14, Fig. 94:

B2, 1½-foot level at center of column flange
N1, 4½-foot level at center of column web
N2, 4½-foot level at center of column flange
M1, 7½-foot level at center of column web
M2, 7½-foot level at center of column flange
M3, 7½-foot level at edge of column flange
T2, 10½-foot level at center of column flange

cf. Section VII, paragraph *3b*, and Fig. 31.

The arrows in Figs. 90 to 141 indicate the time of failure of the columns in the fire tests. The arrows in Figs. 142 to 145 indicate the time water was applied to the columns in the fire and water tests.

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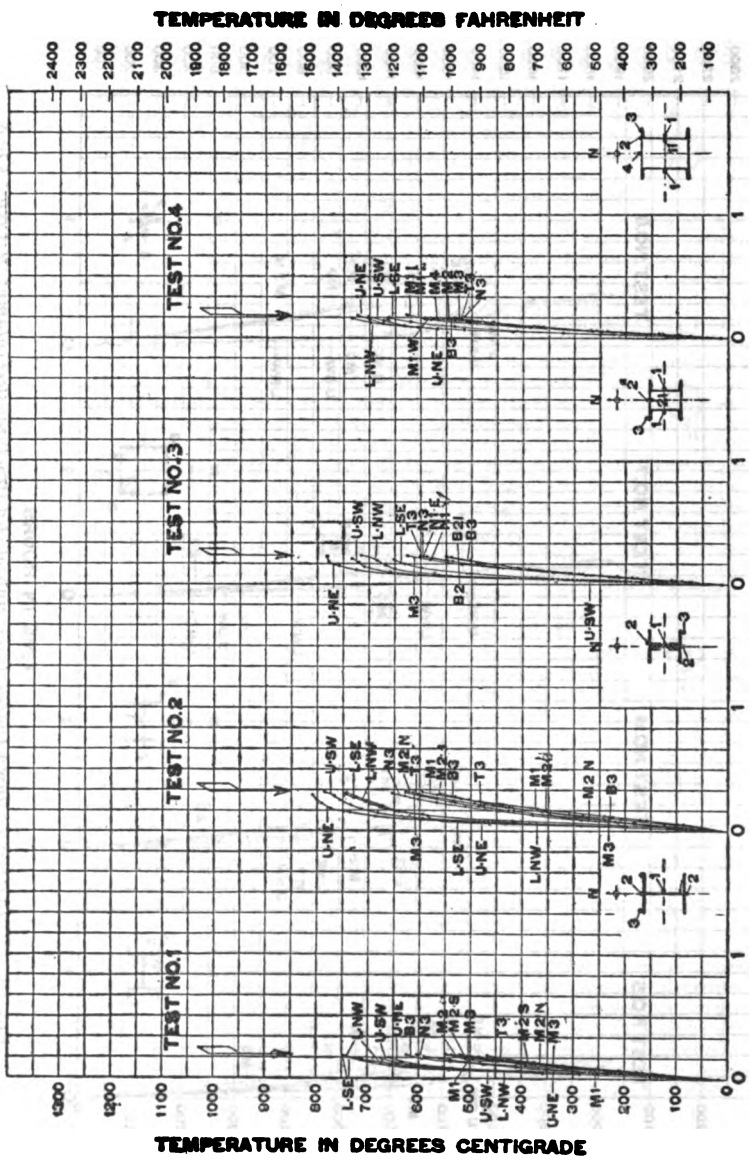

FIG. 90.—*Furnace and column temperatures, unprotected structural-steel columns, test Nos. 1 to 4*

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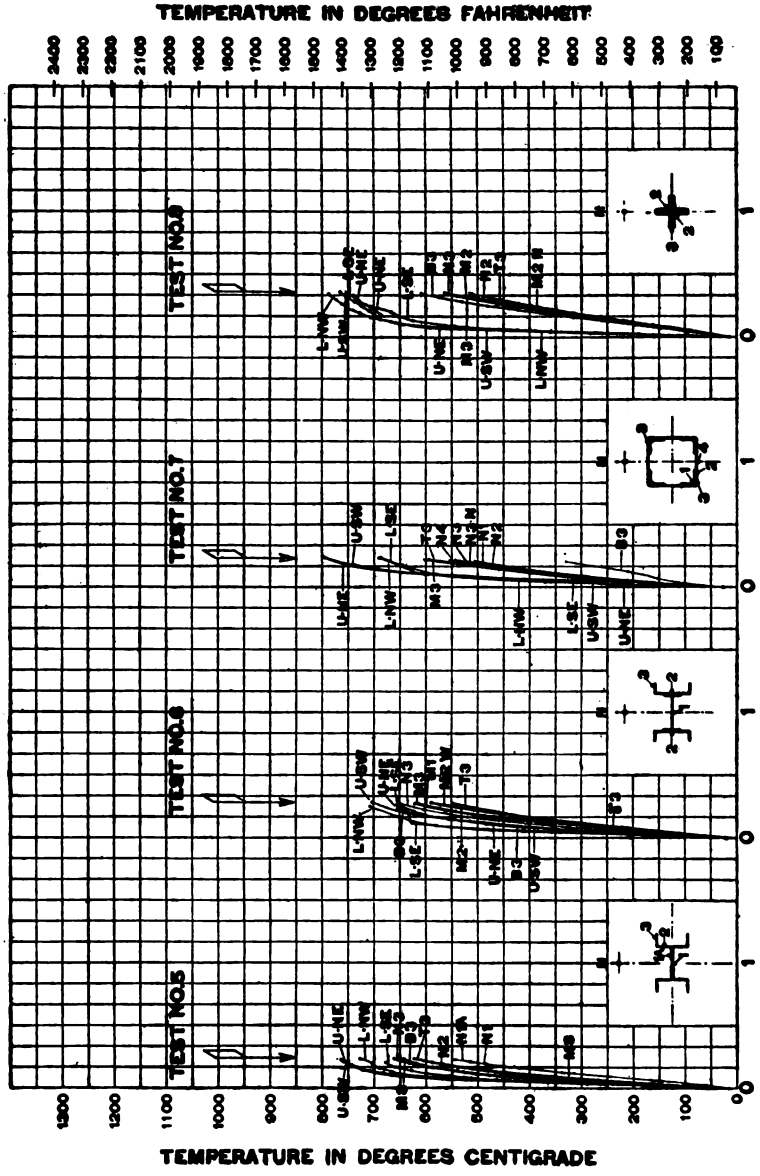

FIG. 91.—*Furnace and column temperatures, unprotected structural-steel columns, test Nos. 5 to 8*

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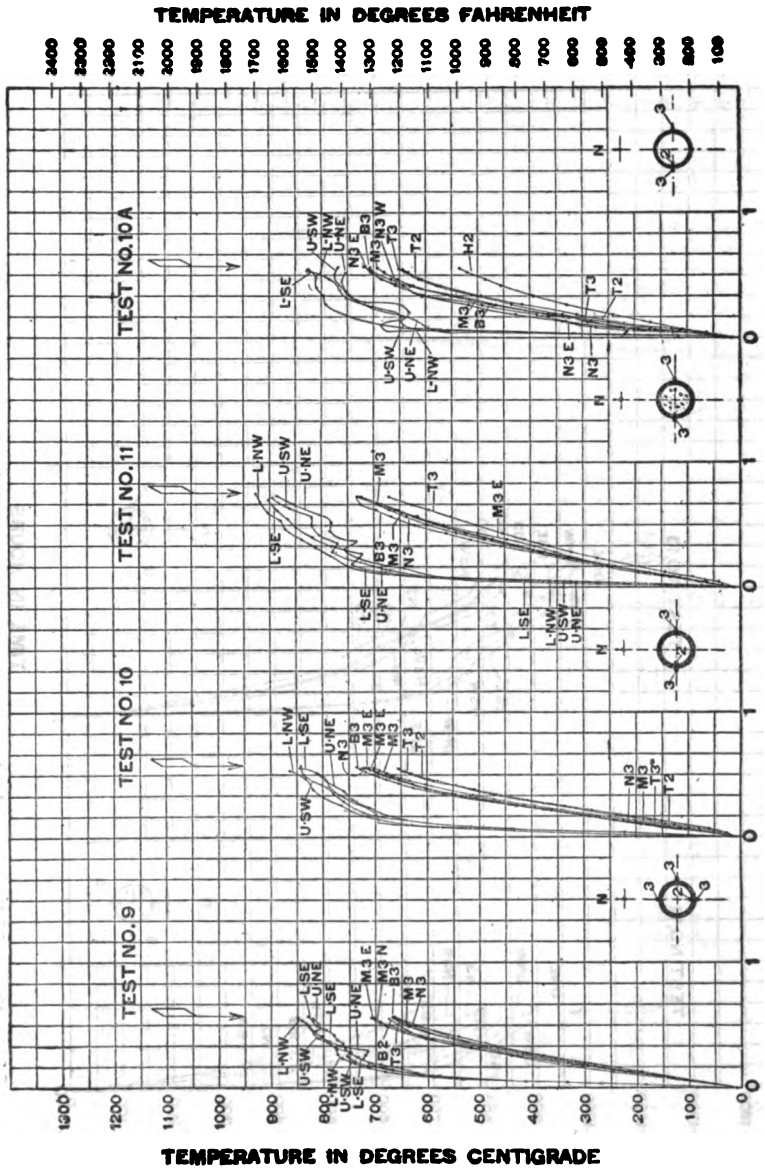

FIG. 92.—*Furnace and column temperatures, unprotected cast-iron columns, test Nos. 9 to 11*

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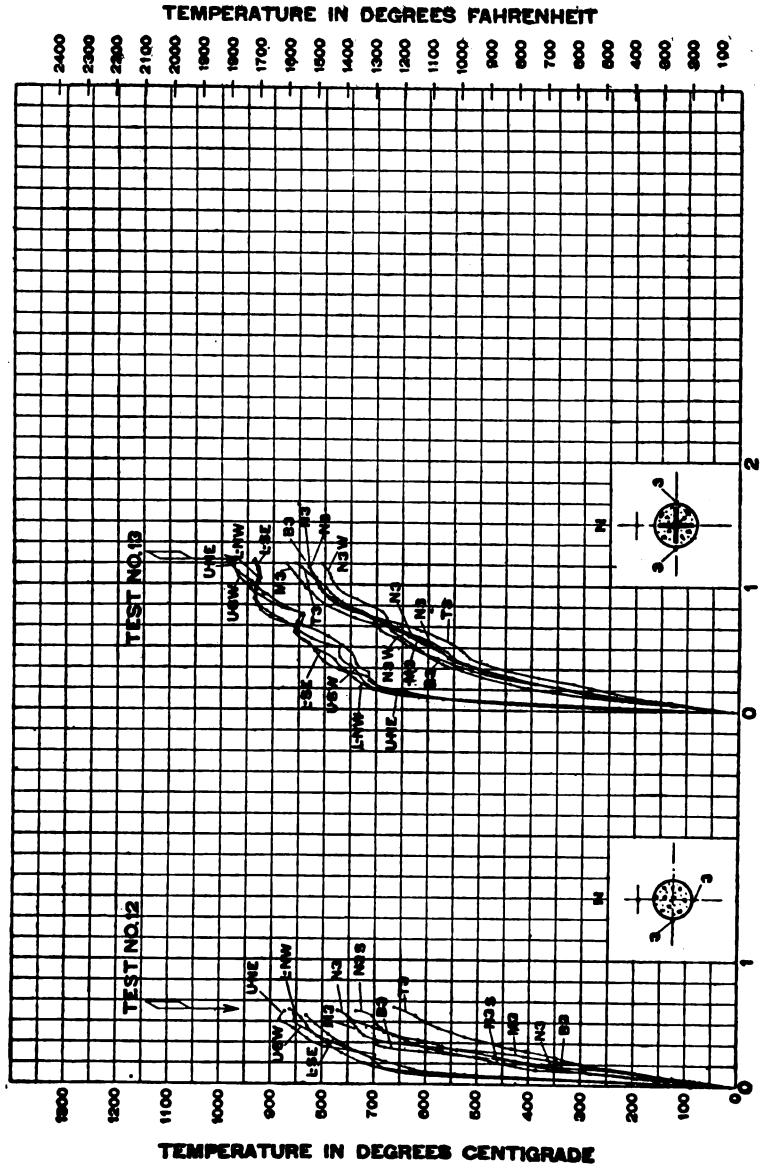

FIG. 93.—*Furnace and column temperatures, unprotected pipe columns, test Nos. 12 and 13*

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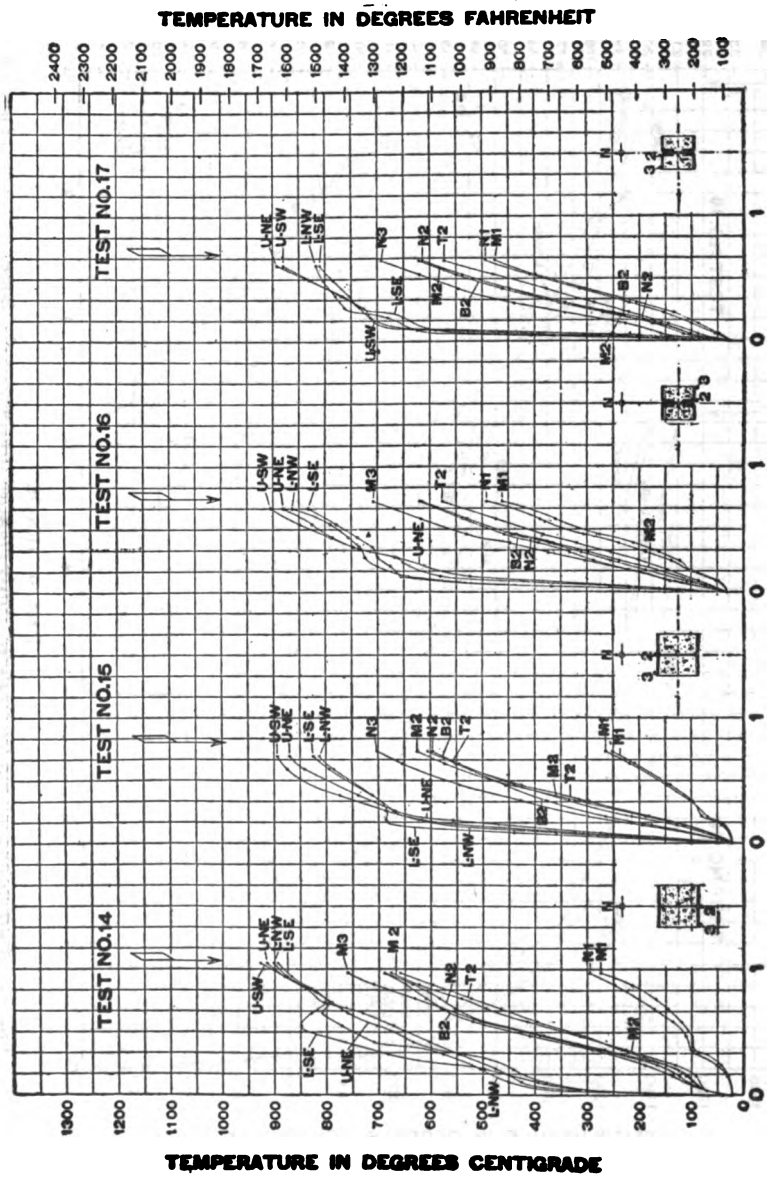

FIG. 94.—*Furnace and column temperatures, partly projected structural-steel columns, test Nos. 14 to 17*

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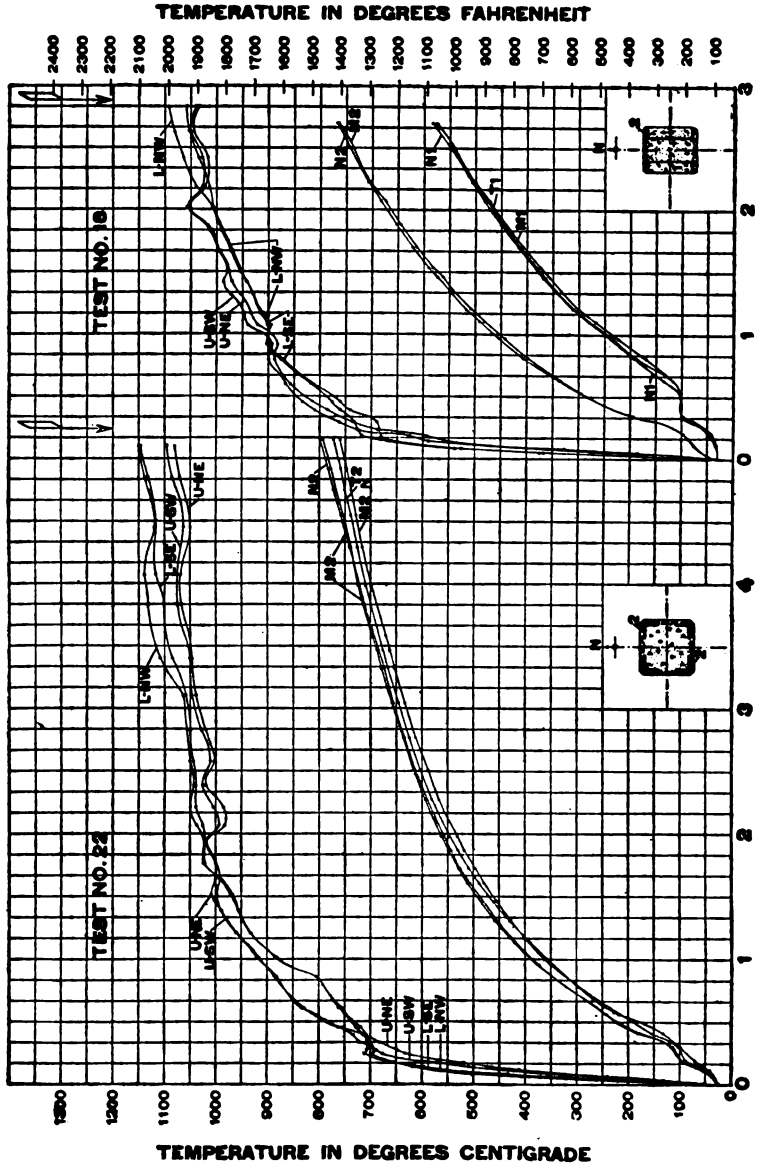

FIG. 95.—*Furnace and column temperatures, partly protected structural-steel columns, test Nos. 18 and 22*

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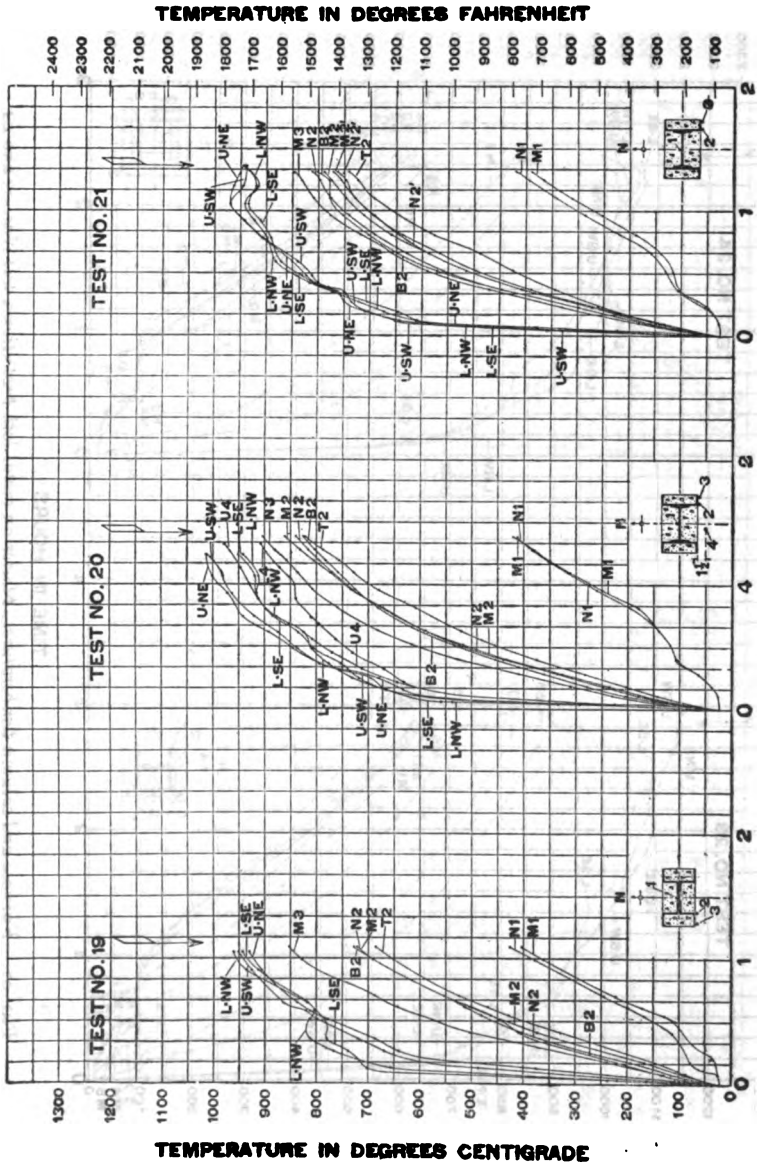

FIG. 96.—*Furnace and column temperatures, partly protected structural-steel columns, test Nos. 19, 20, and 21*

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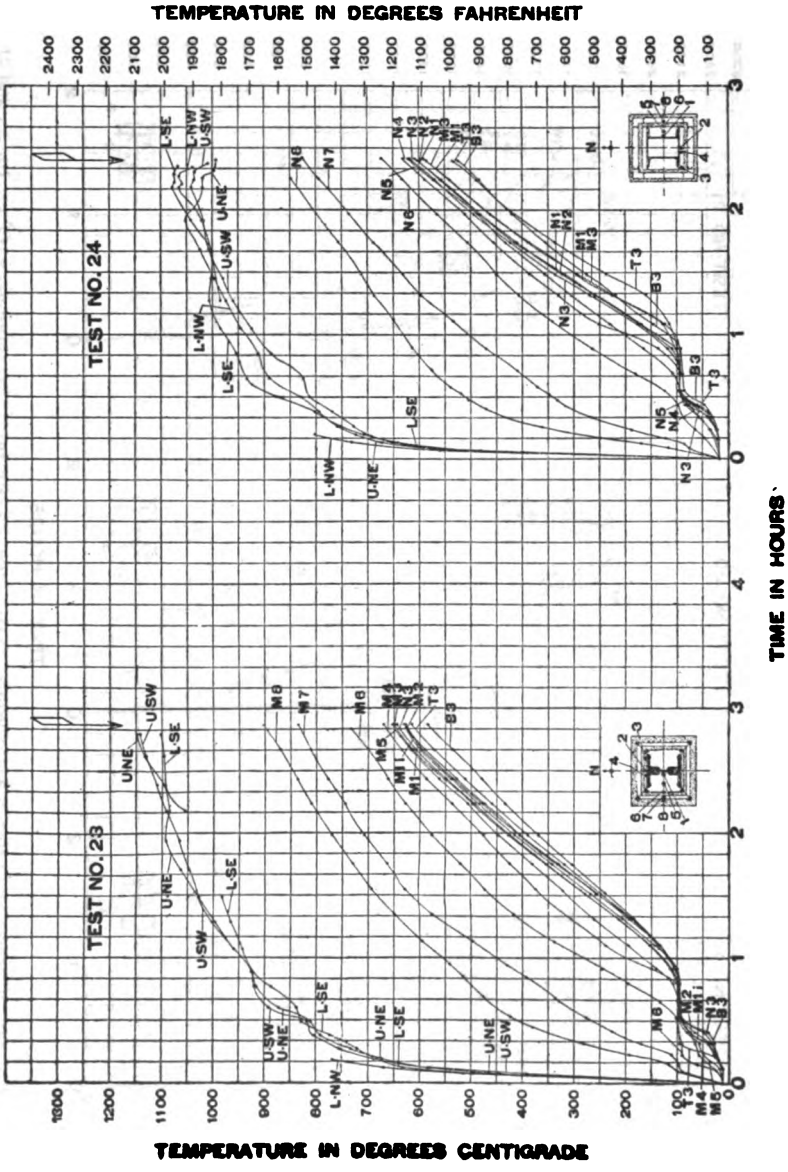

FIG. 97.—*Furnace and column temperatures, plaster on metal-lath protections, test Nos. 23 and 24*

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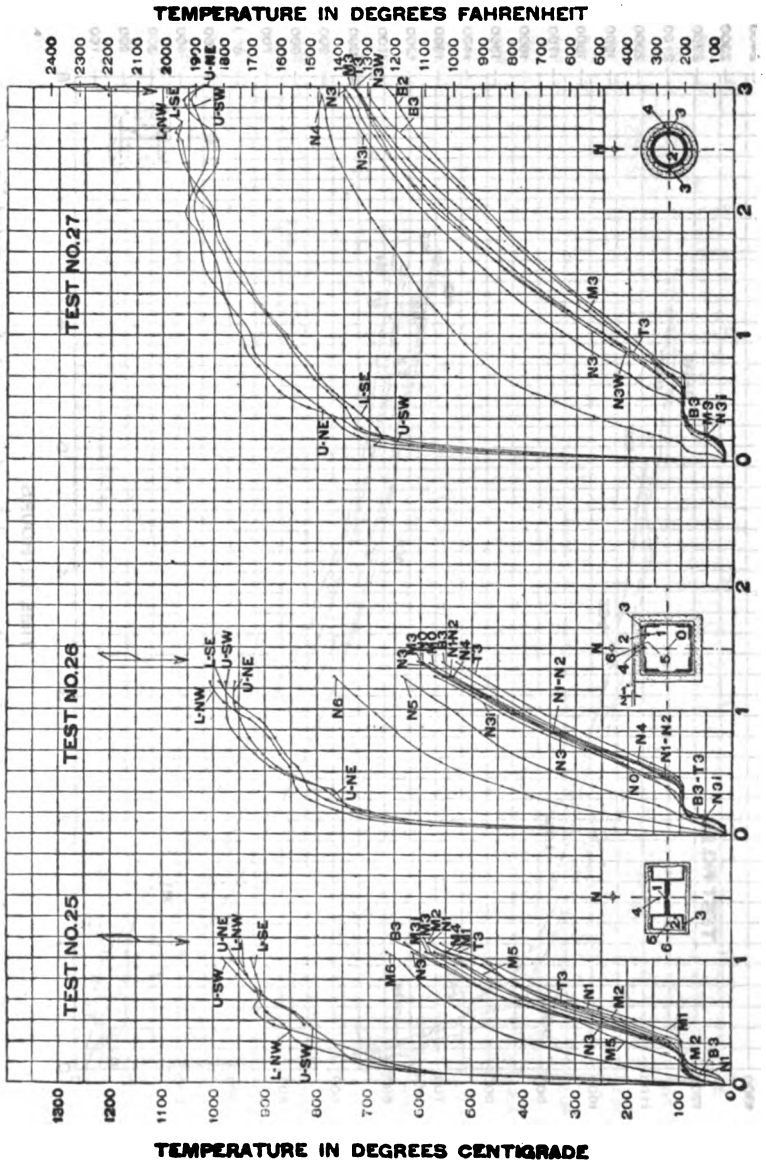

FIG. 98.—*Furnace and column temperatures, plaster on metal-lath protections, test Nos. 25, 26, and 27*

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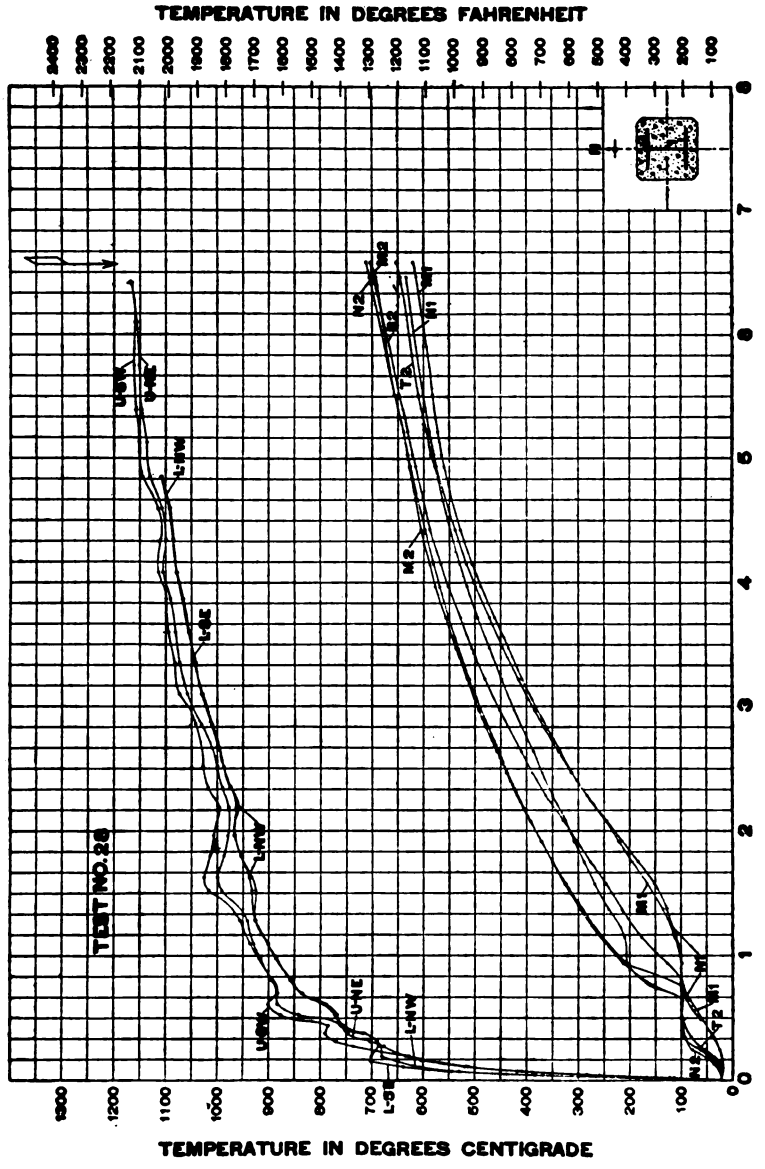

FIG. 99.—*Furnace and column temperatures, 2-inch limestone concrete protection, test No. 28*

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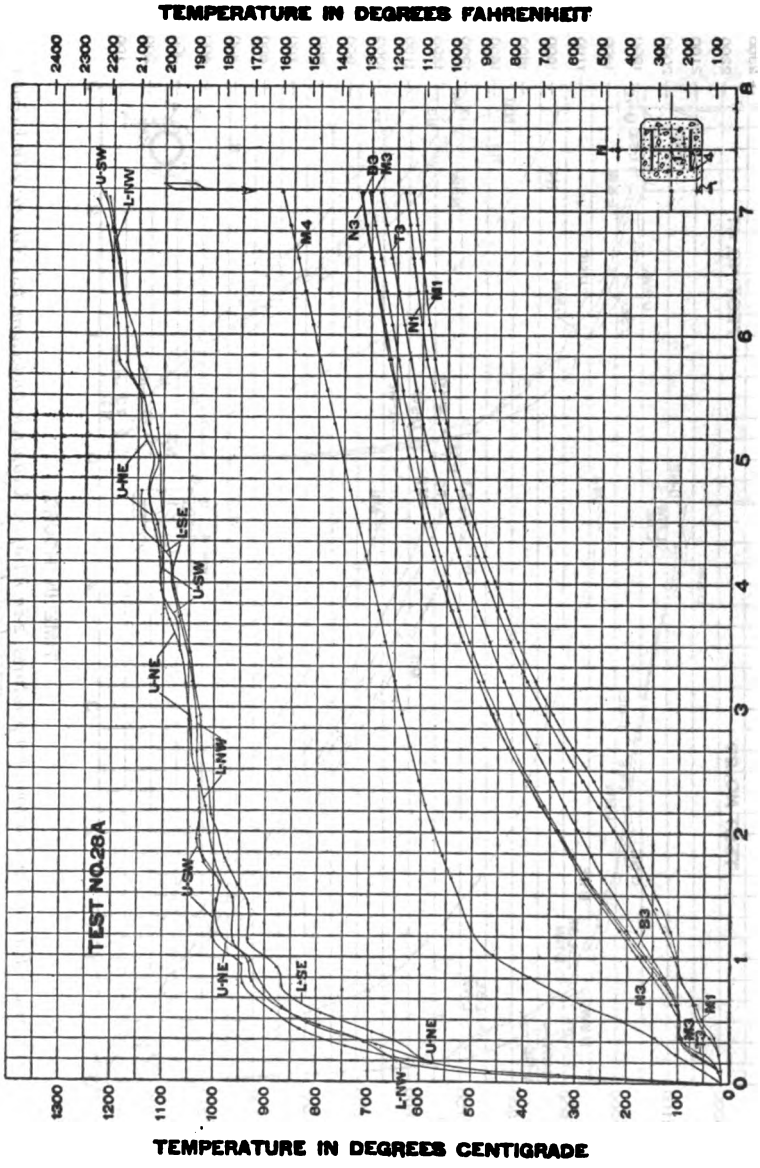

FIG. 100.—*Furnace and column temperatures 2-inch limestone concrete protection, test No. 28A*

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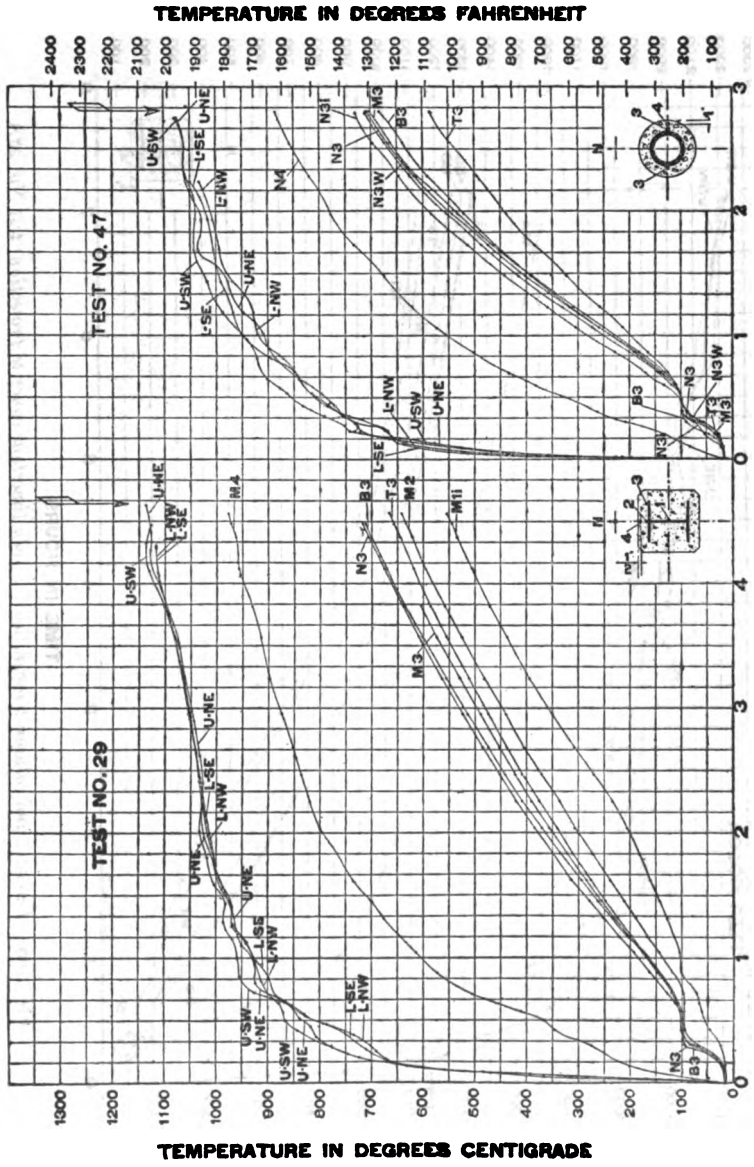

FIG. 101.—*Furnace and column temperatures, 2-inch trap-rock concrete protection, test No. 29; 2-inch cinder concrete protection, test No. 47*

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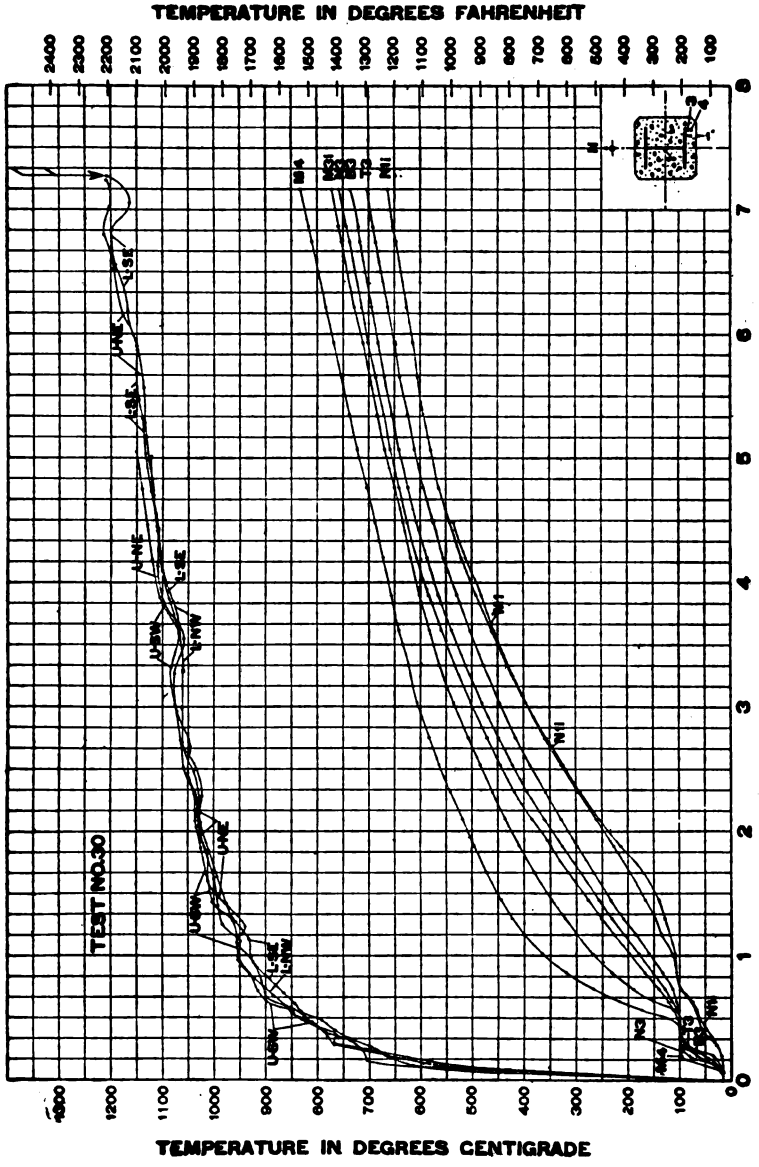

FIG. 102.—*Furnace and column temperatures, 2-inch Joliet gravel concrete protection, test No. 30*

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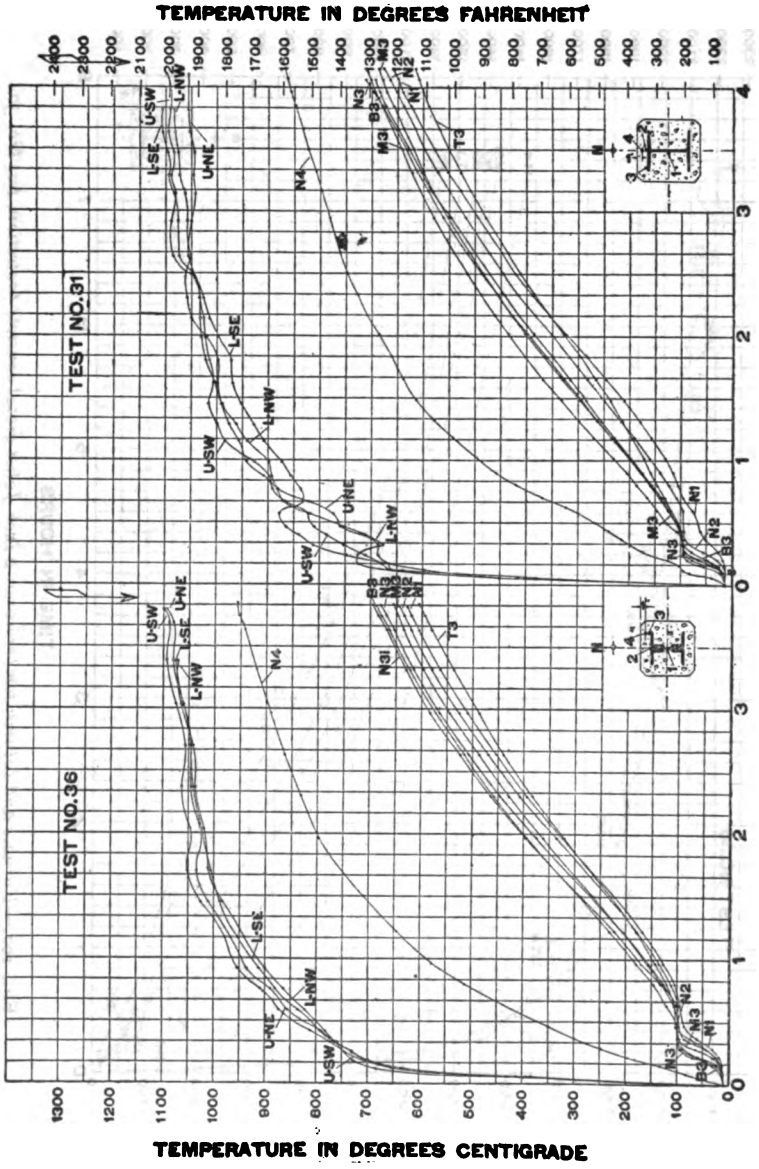

FIG. 103.—*Furnace and column temperatures, 2-inch sandstone concrete protection, test No. 31; 2-inch trap-rock concrete protection, test No. 36*

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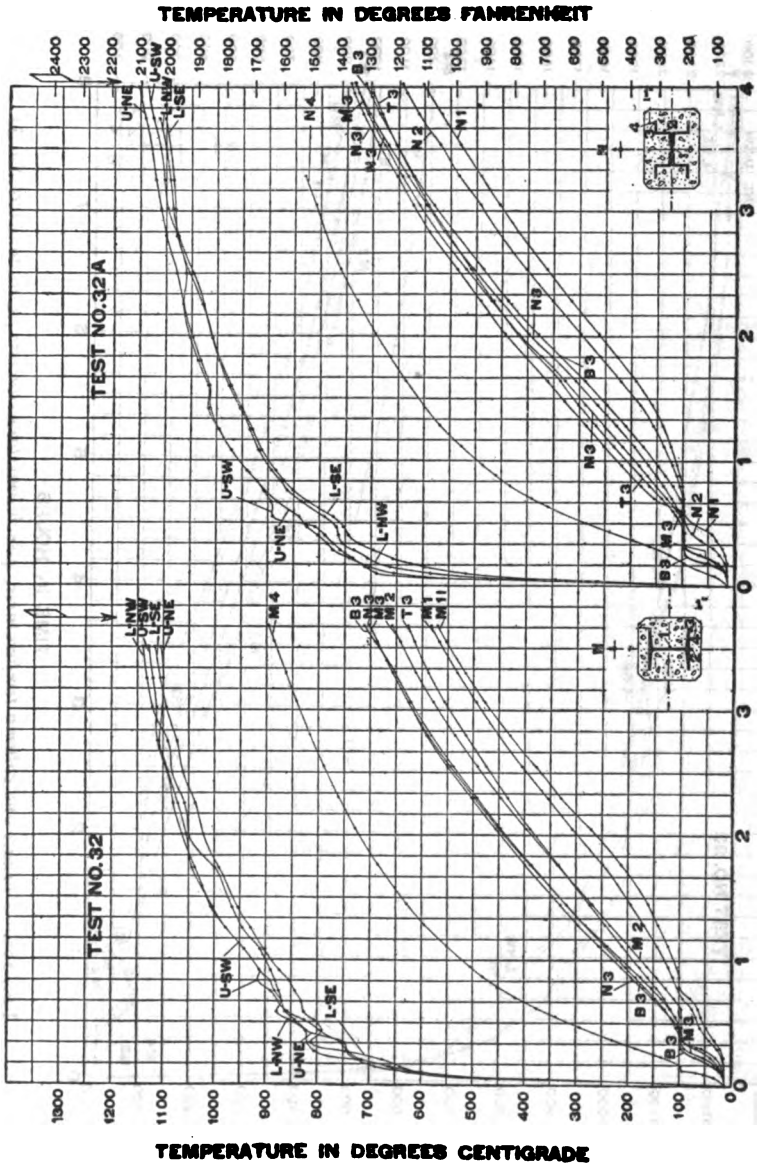

FIG. 104.—*Furnace and column temperatures, 2-inch cinder concrete protections, test Nos. 32 and 32A*

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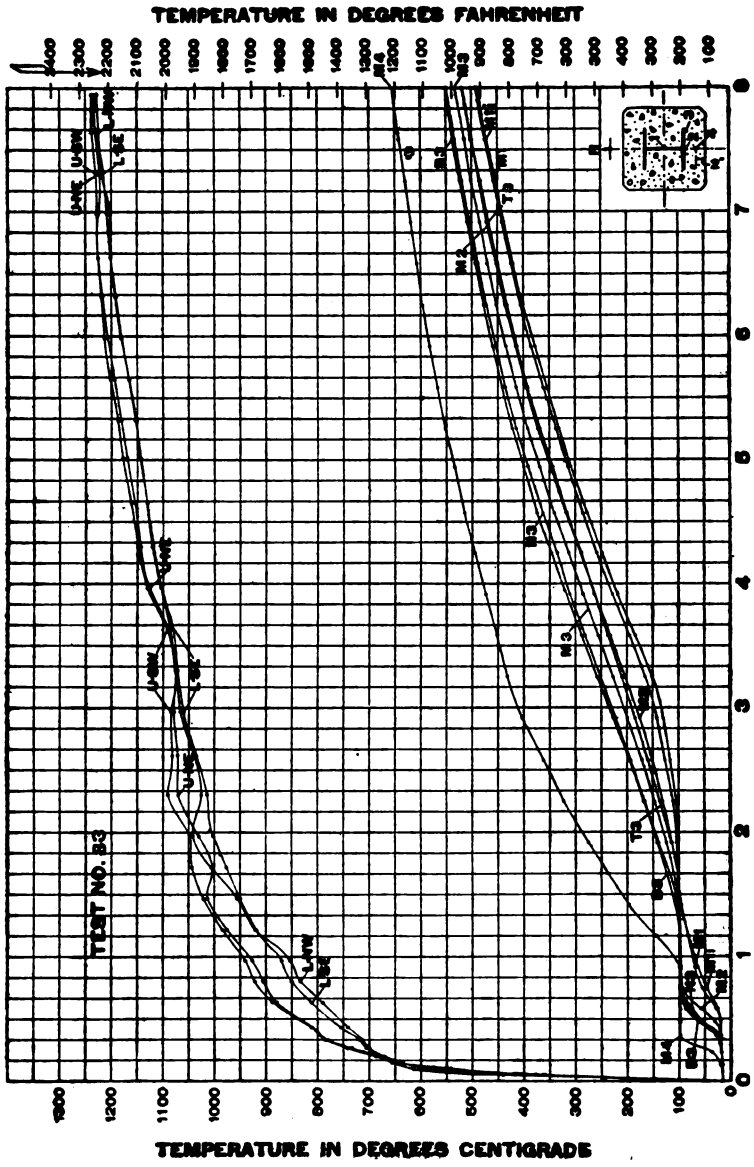

FIG. 105.—*Furnace and column temperatures, 4-inch limestone concrete protection, test No. 33*

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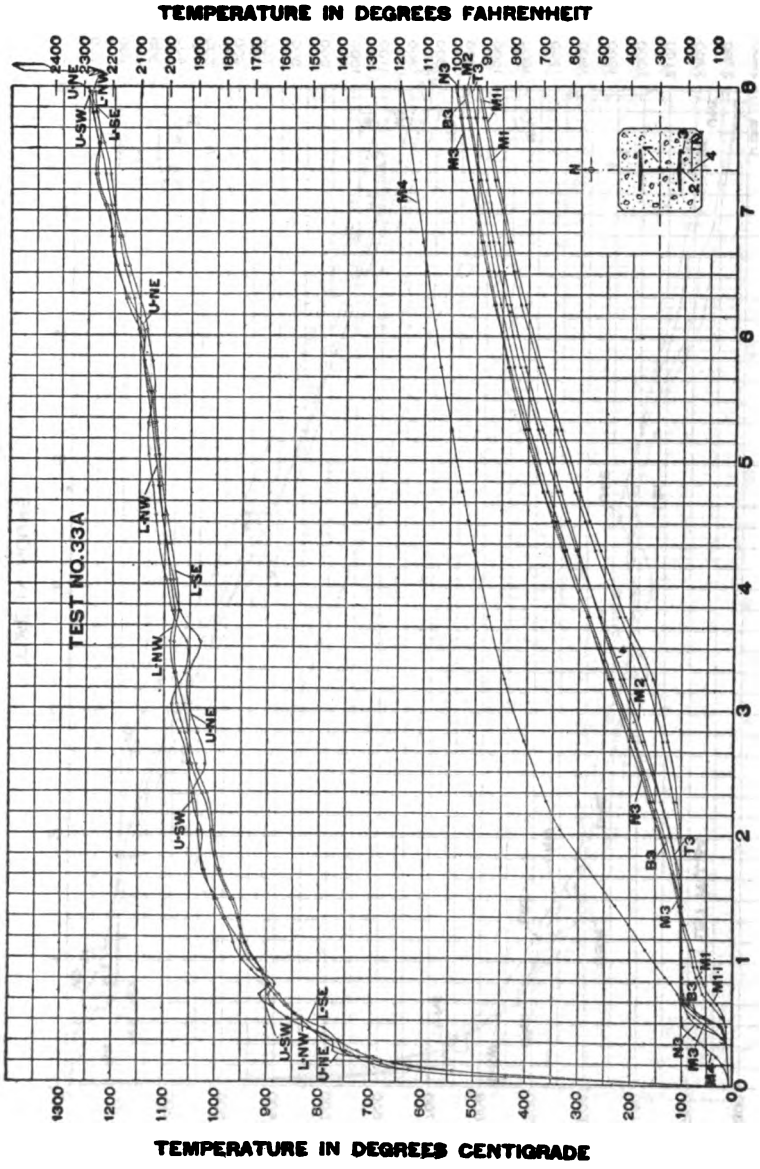

FIG. 106.—*Furnace and column temperatures, 4-inch limestone concrete protection, test No. 33A*

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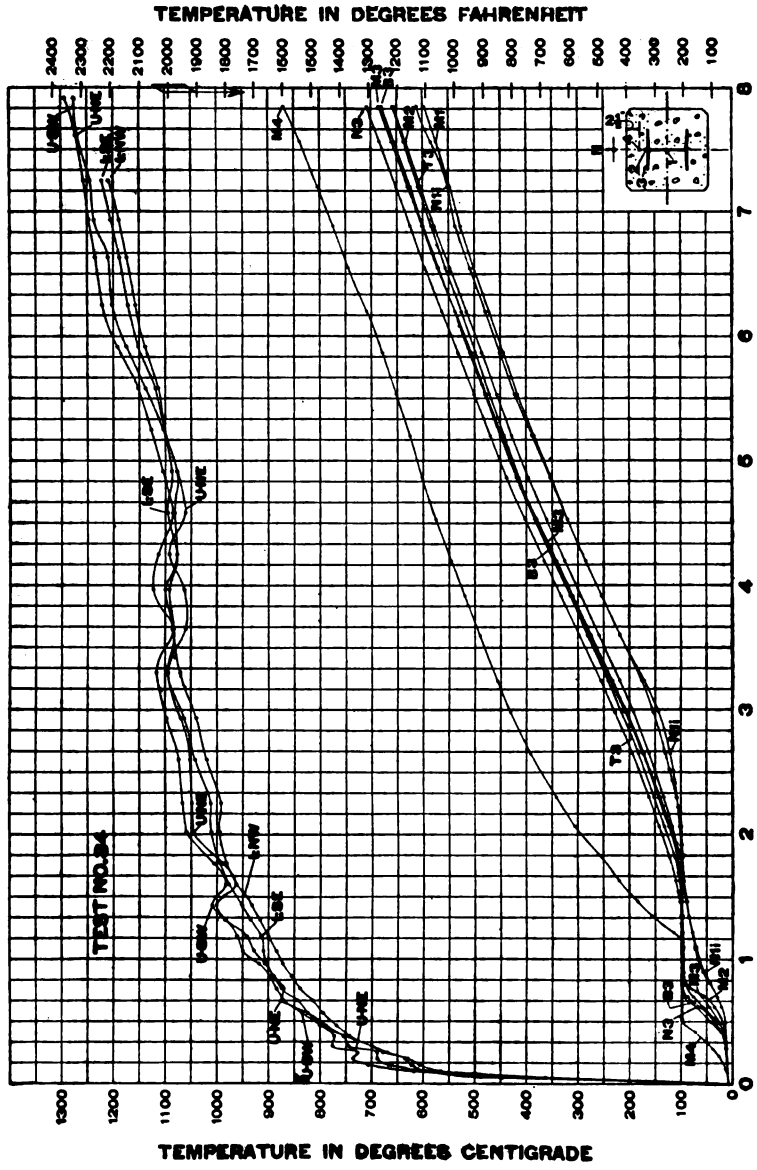

FIG. 107.—*Furnace and column temperatures, 4-inch granite concrete protection, test No. 34*

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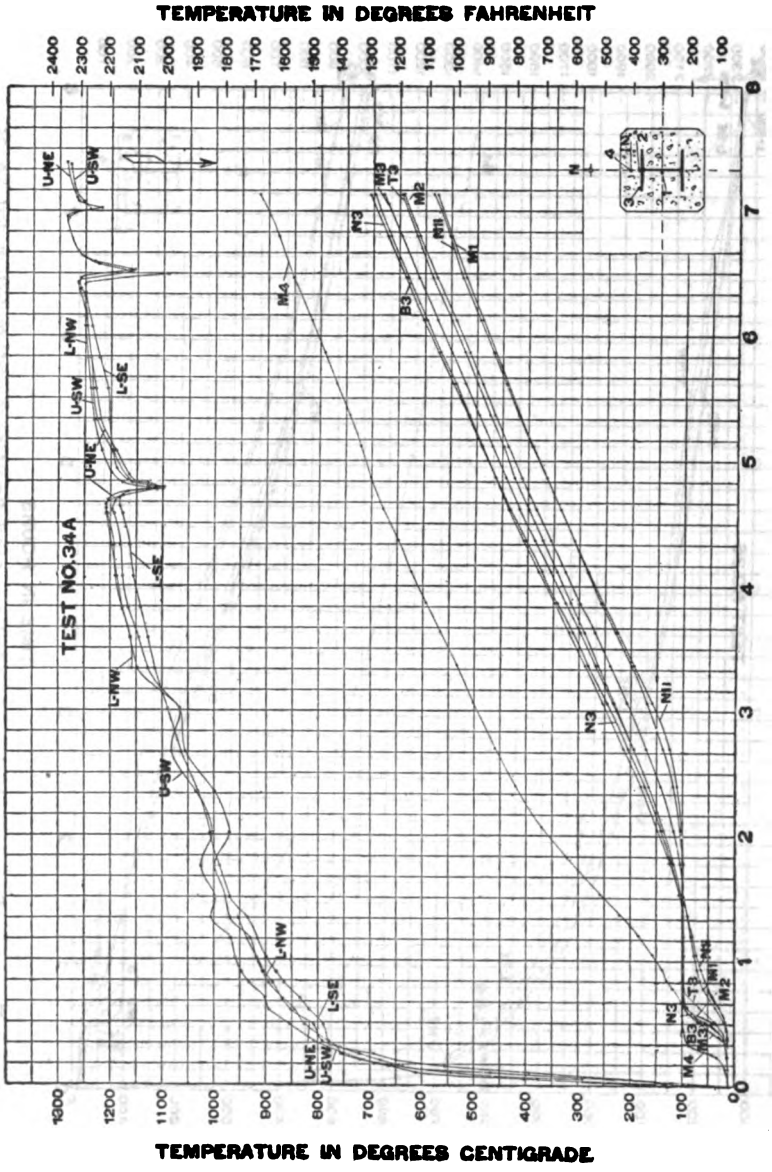

FIG. 108.—*Furnace and column temperatures, 4-inch granite concrete protection, test No. 34A*

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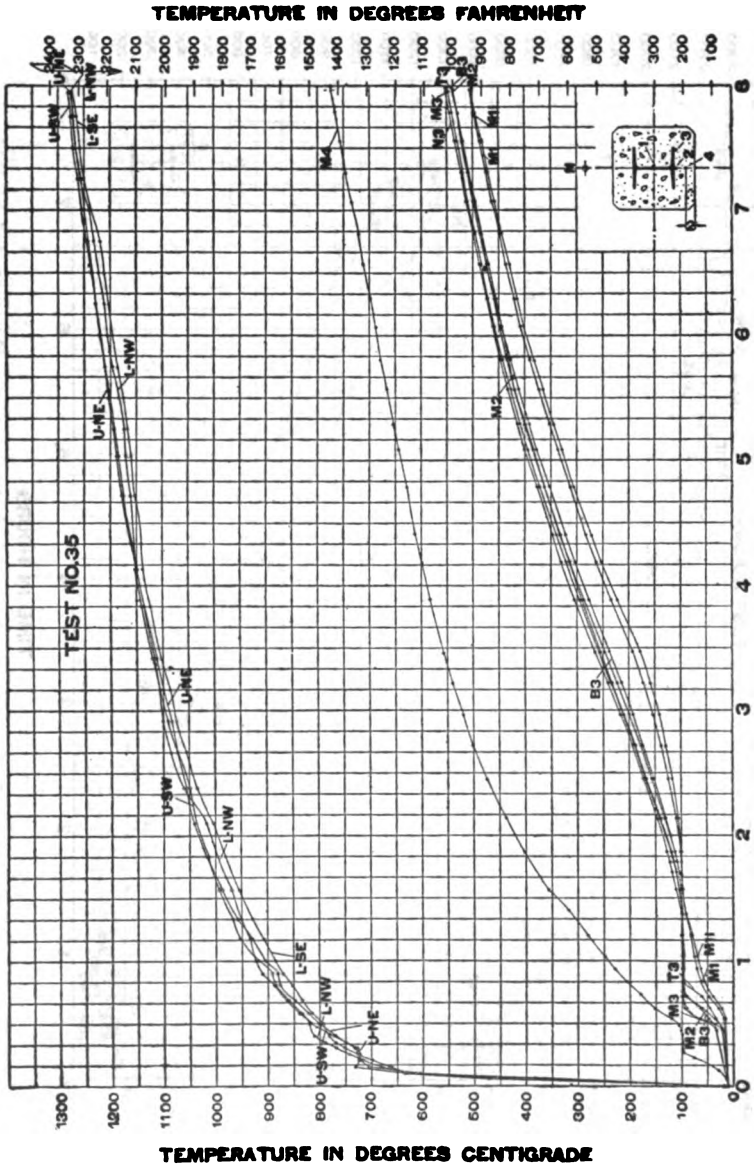

FIG. 109.—*Furnace and column temperatures, 4-inch limestone concrete protection, test No. 35*

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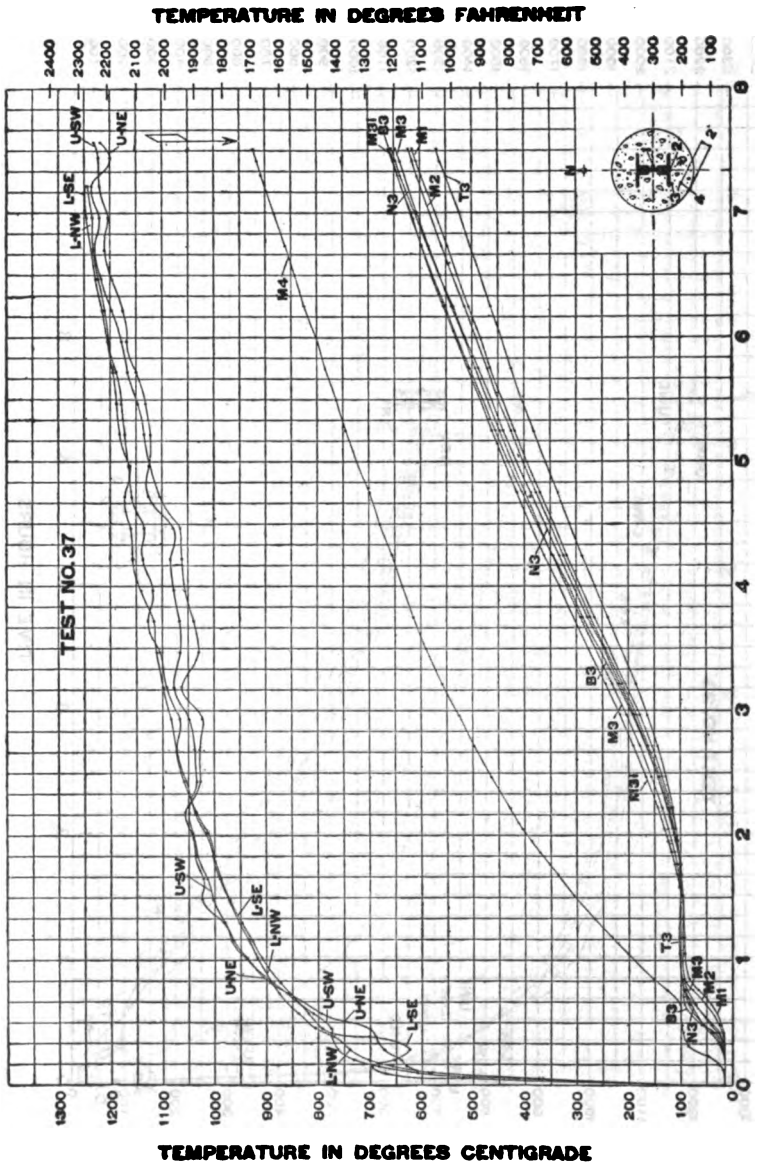

FIG. 110.—*Furnace and column temperatures, 4-inch trap-rock concrete protection, test No. 37*

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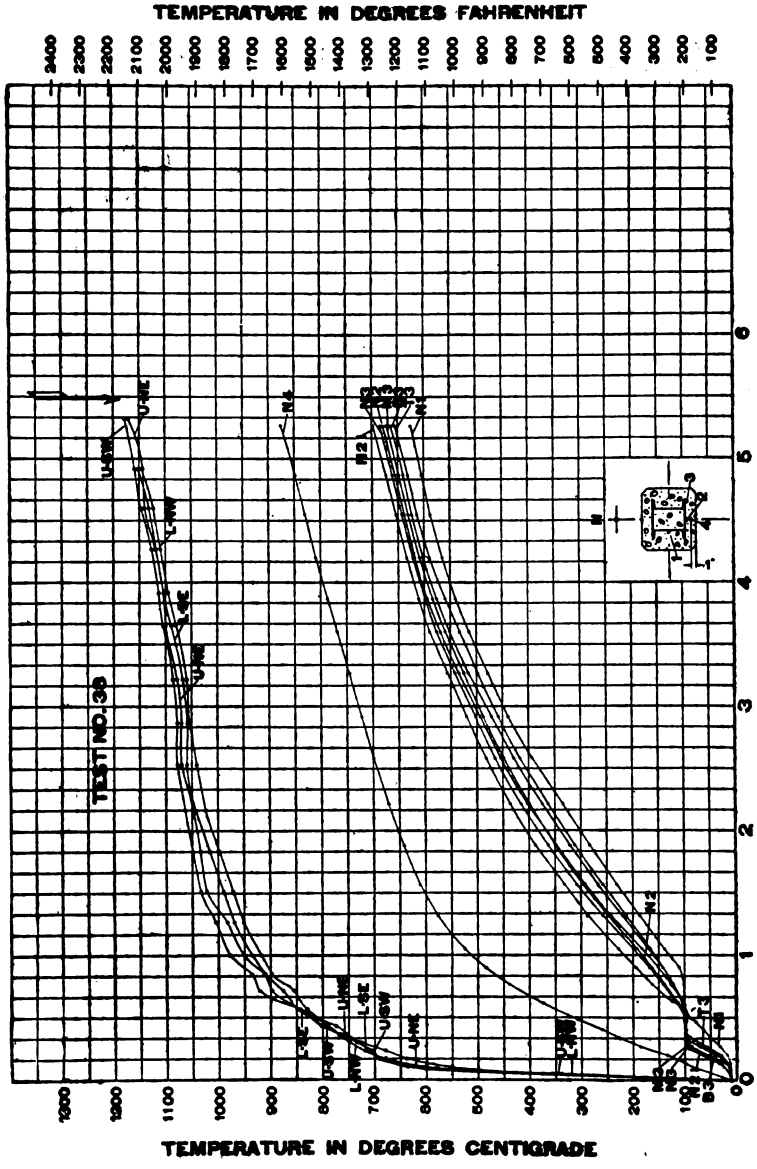

FIG. 111.—*Furnace and column temperatures, 2-inch Joliet gravel concrete protection, test No. 38.*

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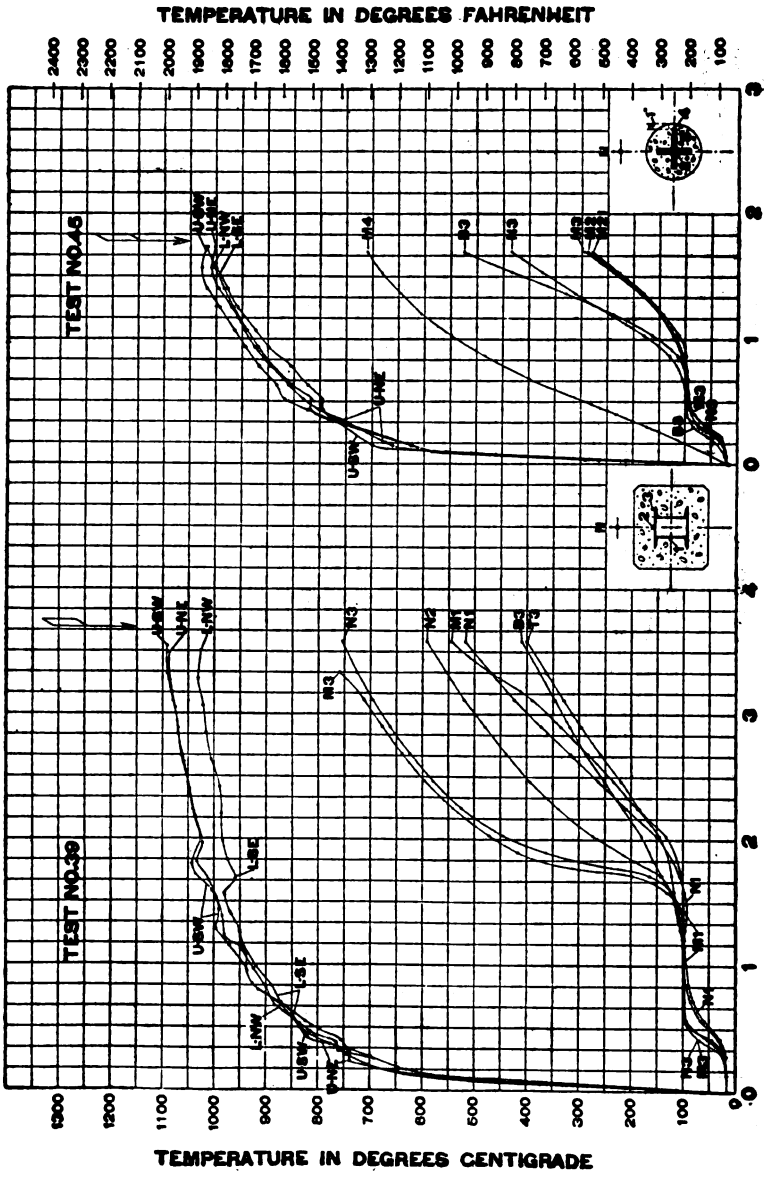

FIG. 112.—*Furnace and column temperatures, 4-inch Meramec River gravel concrete protection, test No. 39; 2-inch Meramec River gravel concrete protection, test No. 45*

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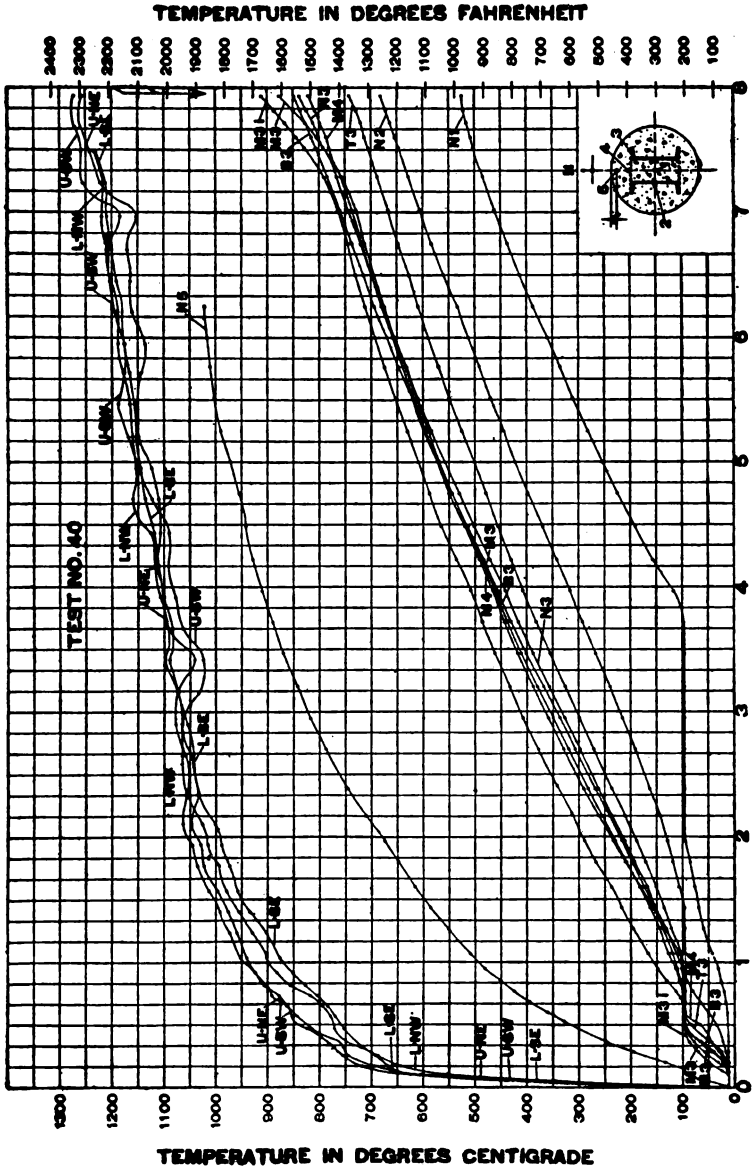

FIG. 113.—*Furnace and column temperatures, 2-inch trap-rock concrete protection, test No. 40*

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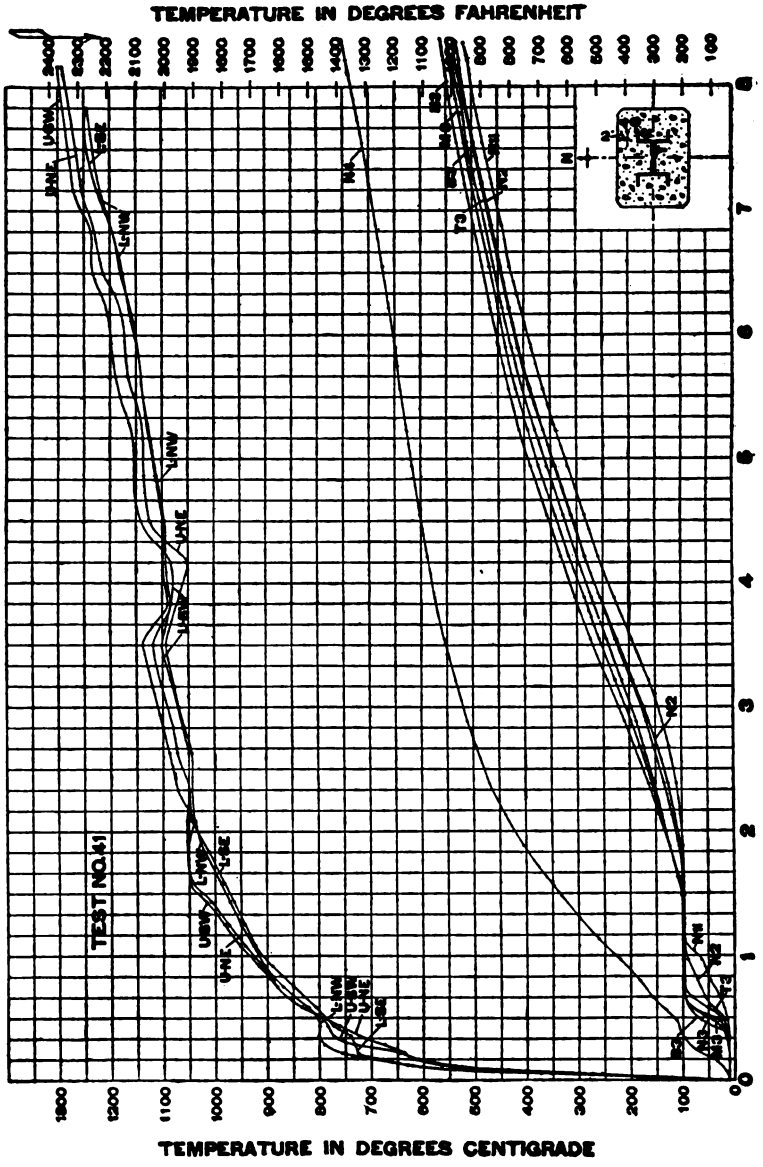

FIG. 114.—*Furnace and column temperatures, 4-inch limestone concrete protection, test No. 41*

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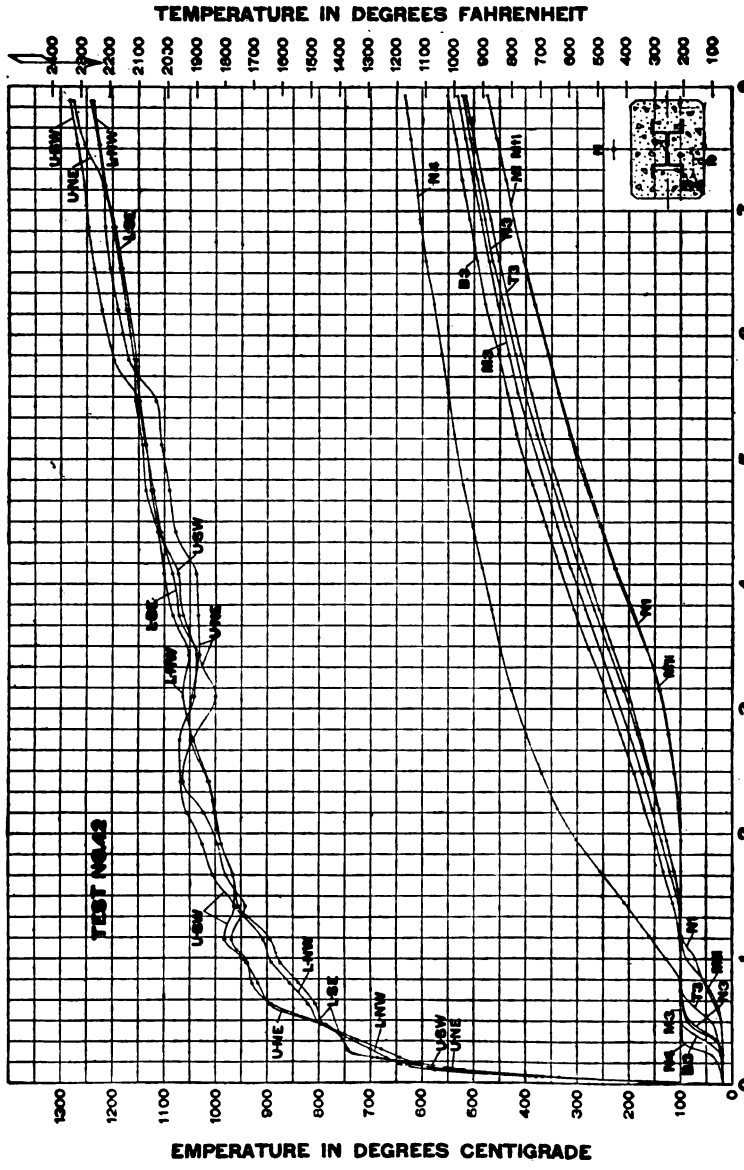

FIG. 115.—*Furnace and column temperatures, 4-inch limestone concrete protection, test No. 42*

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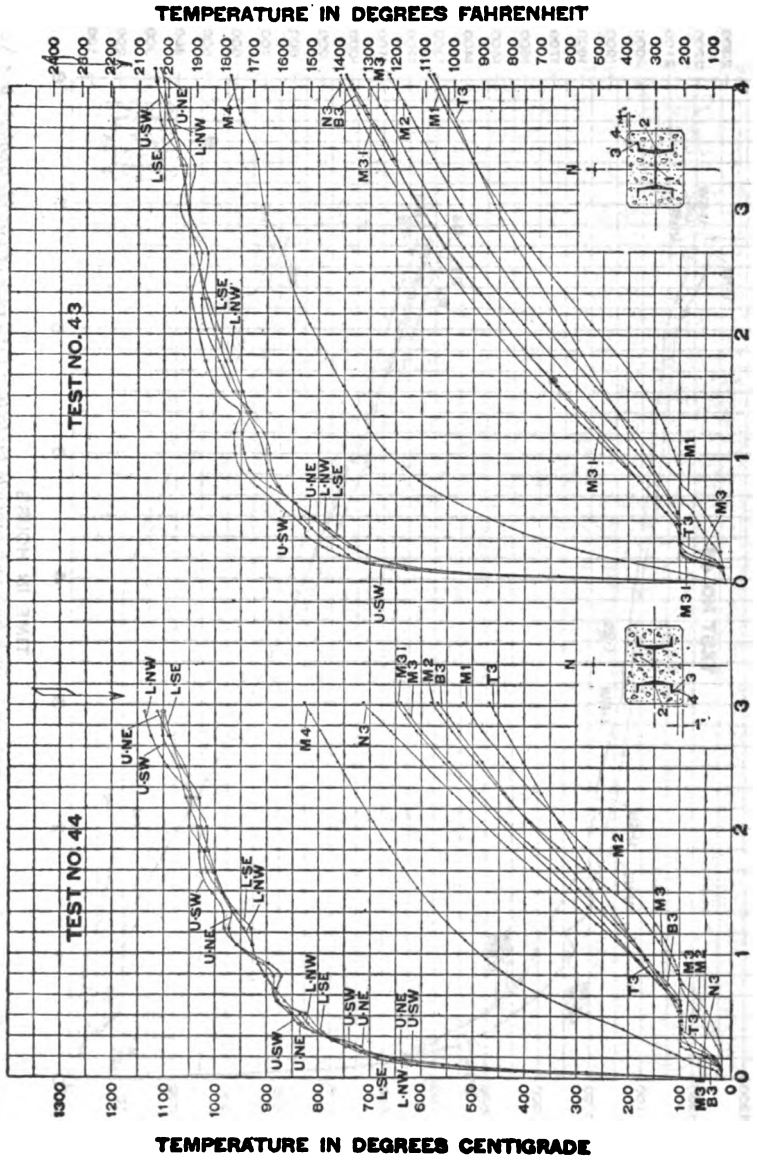

FIG. 116.—*Furnace and column temperatures, 2-inch sandstone concrete protections, test Nos. 43 and 44*

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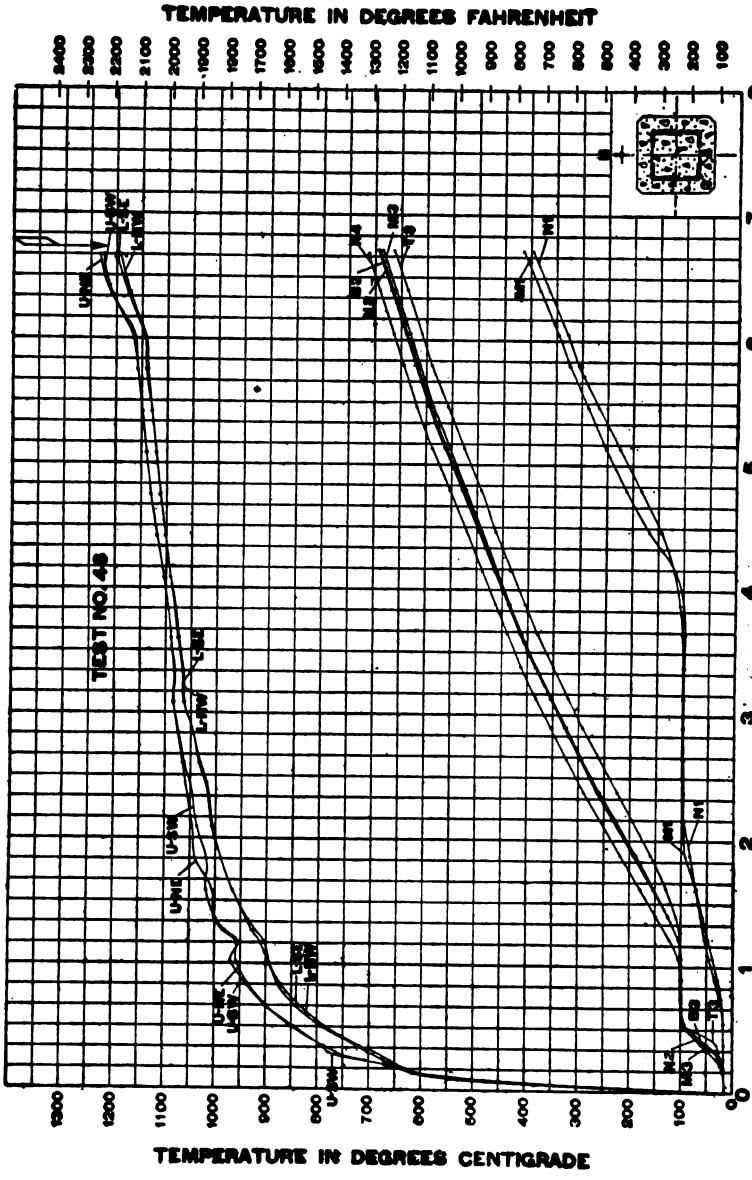

FIG. 117.—*Furnace and column temperatures, 2-inch trap-rock concrete protection (3½-inch of outside angles), test No. 46*

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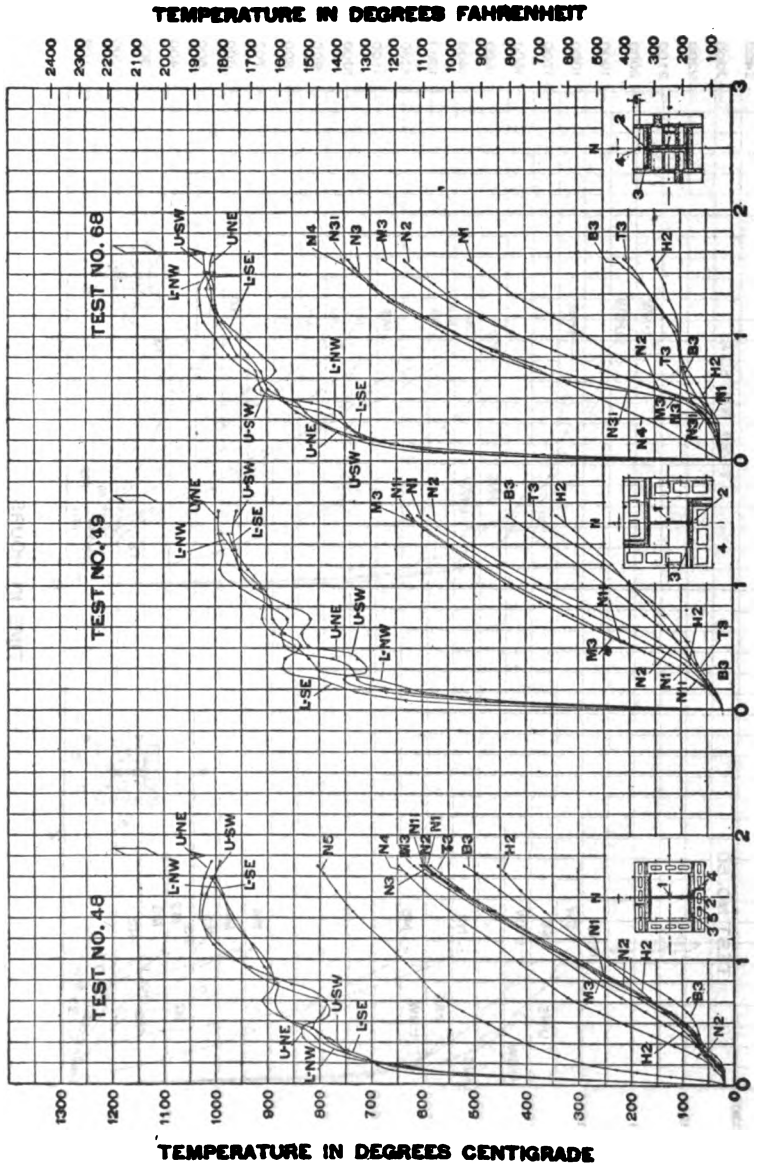

FIG. 118.—*Furnace and column temperatures, semifire-clay tile protections, test Nos. 48 and 49; 2¼-inch surface clay brick protection, test No. 68*

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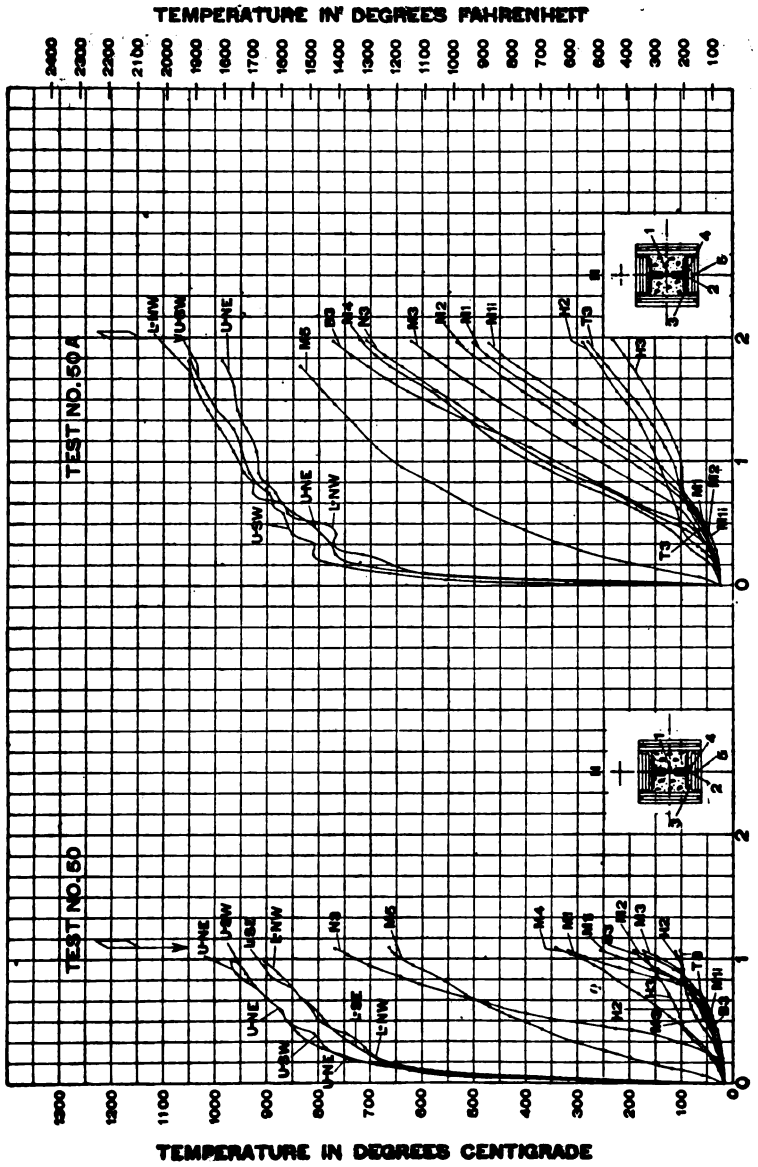

FIG. 119.—*Furnace and column temperatures, surface-clay tile protections, test Nos. 50 and 50A*

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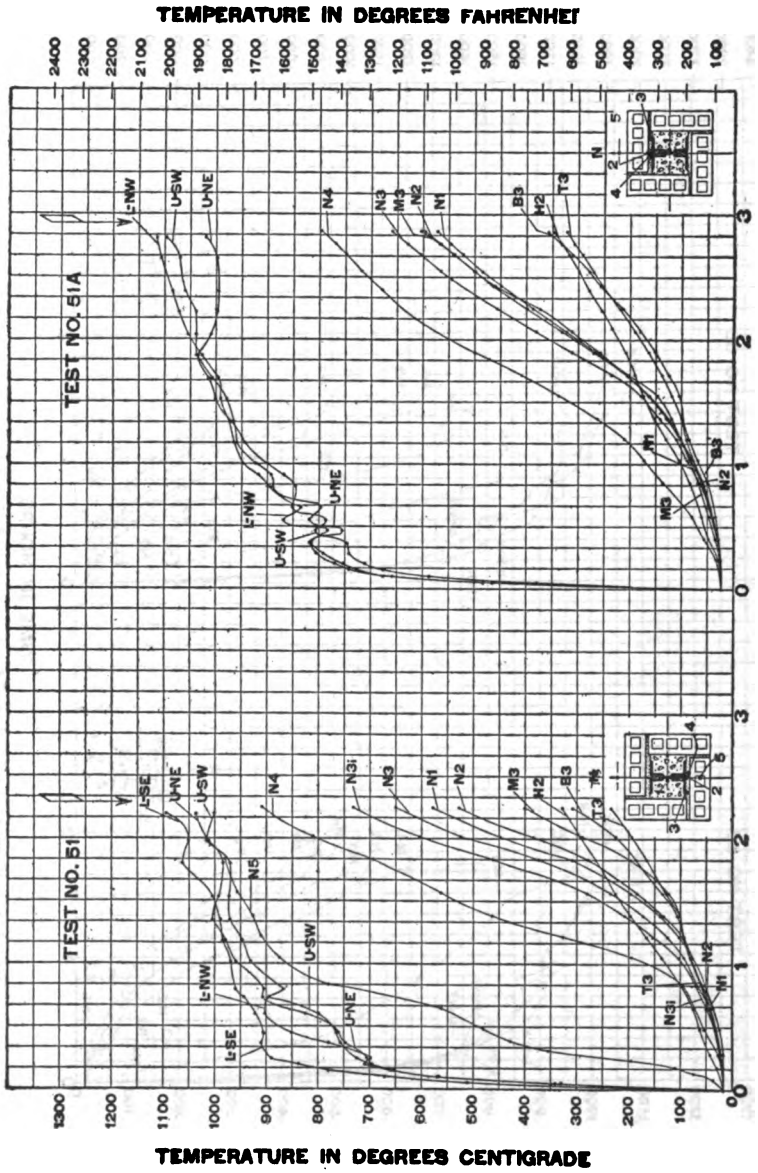

FIG. 120.—*Furnace and column temperatures, surface-clay tile protections, test Nos. 51 and 51A*

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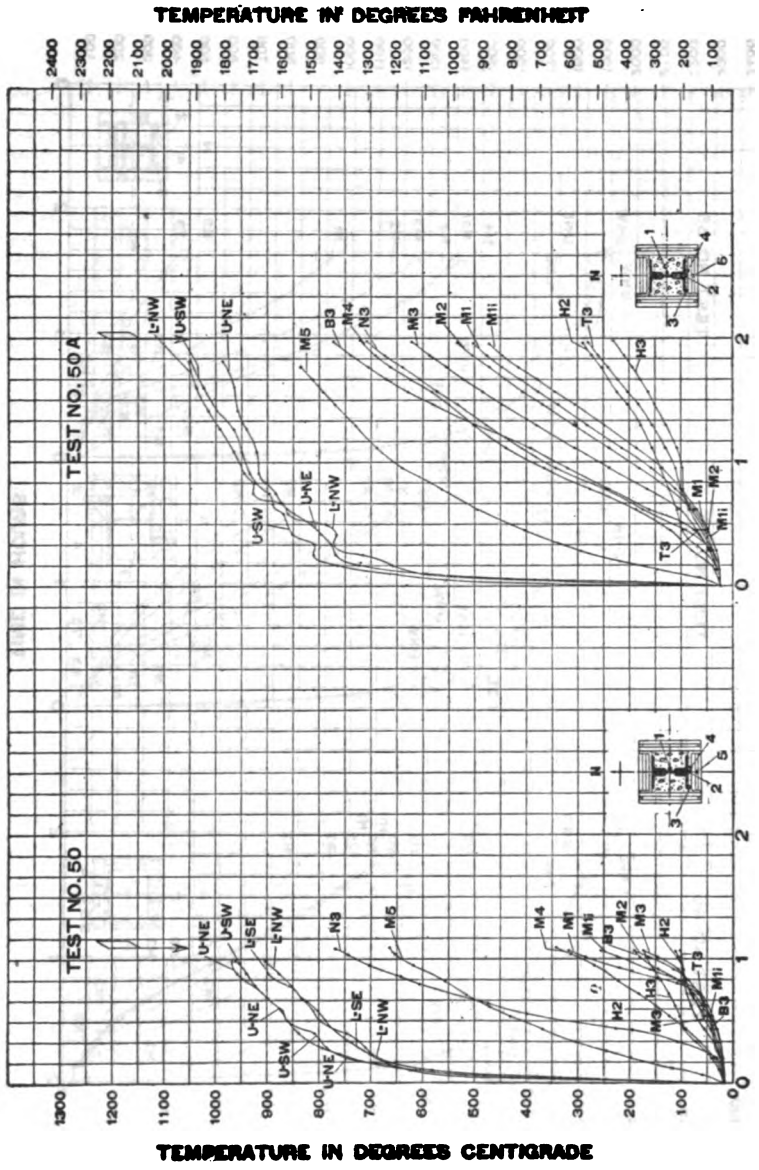

FIG. 119.—*Furnace and column temperatures, surface-clay tile protections, test Nos. 50 and 50A*

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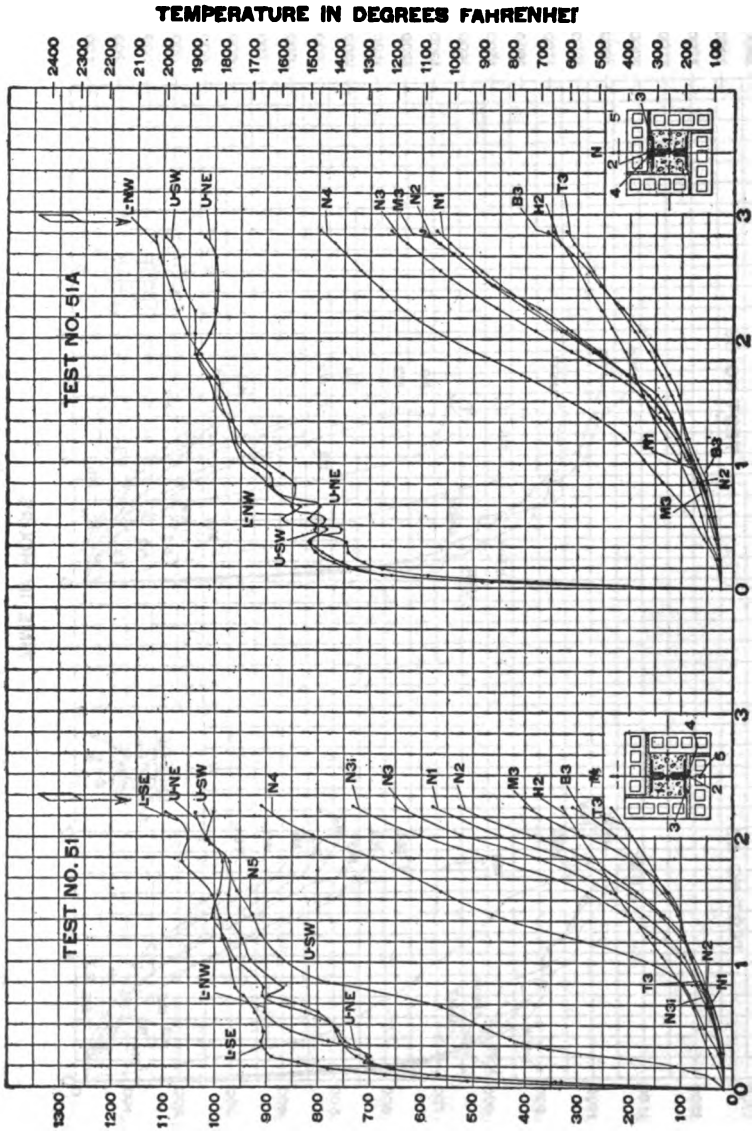

FIG. 120.—*Furnace and column temperatures, surface-clay tile protections, test Nos. 51 and 51A*

20184°—21——19

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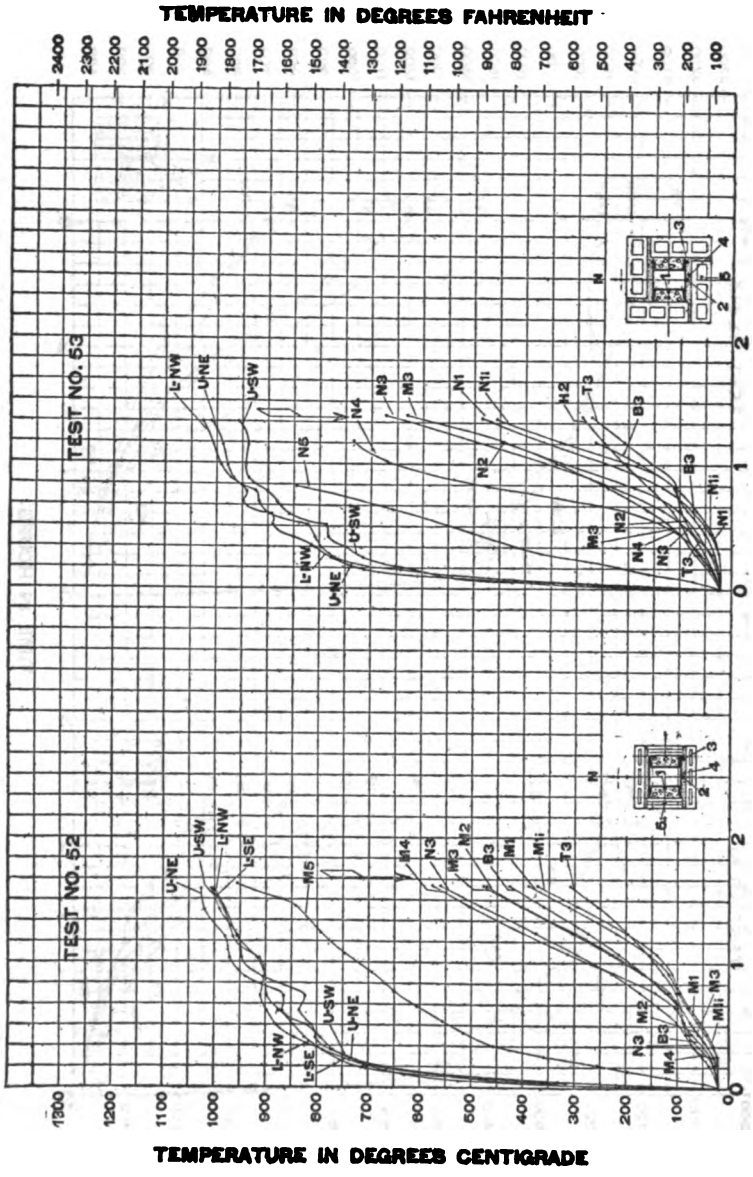

FIG. 121.—*Furnace and column temperatures, shale tile protections, test Nos. 52 and 53*

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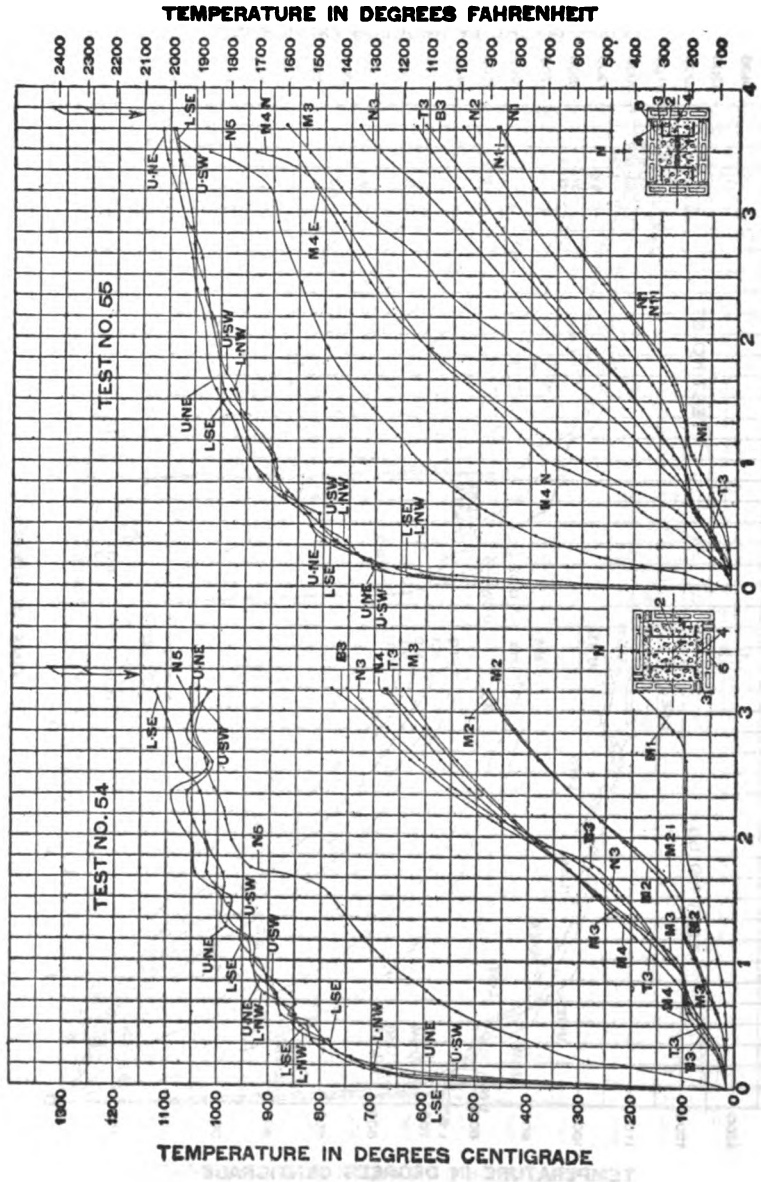

FIG. 122.—*Furnace and column temperatures, semi-fire-clay tile protections, test Nos. 54 and 55*

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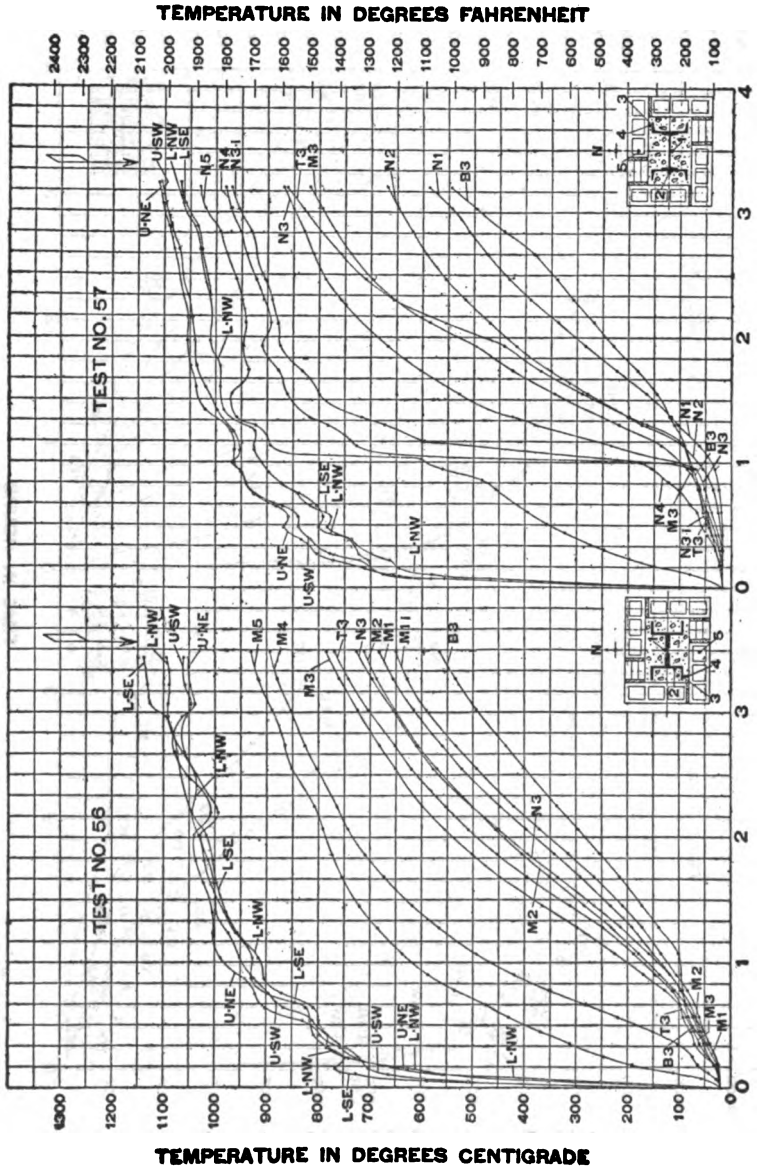

FIG. 123.—*Furnace and column temperatures, semifire-clay tile protection, test No. 56; surface-clay tile protection, test No. 57*

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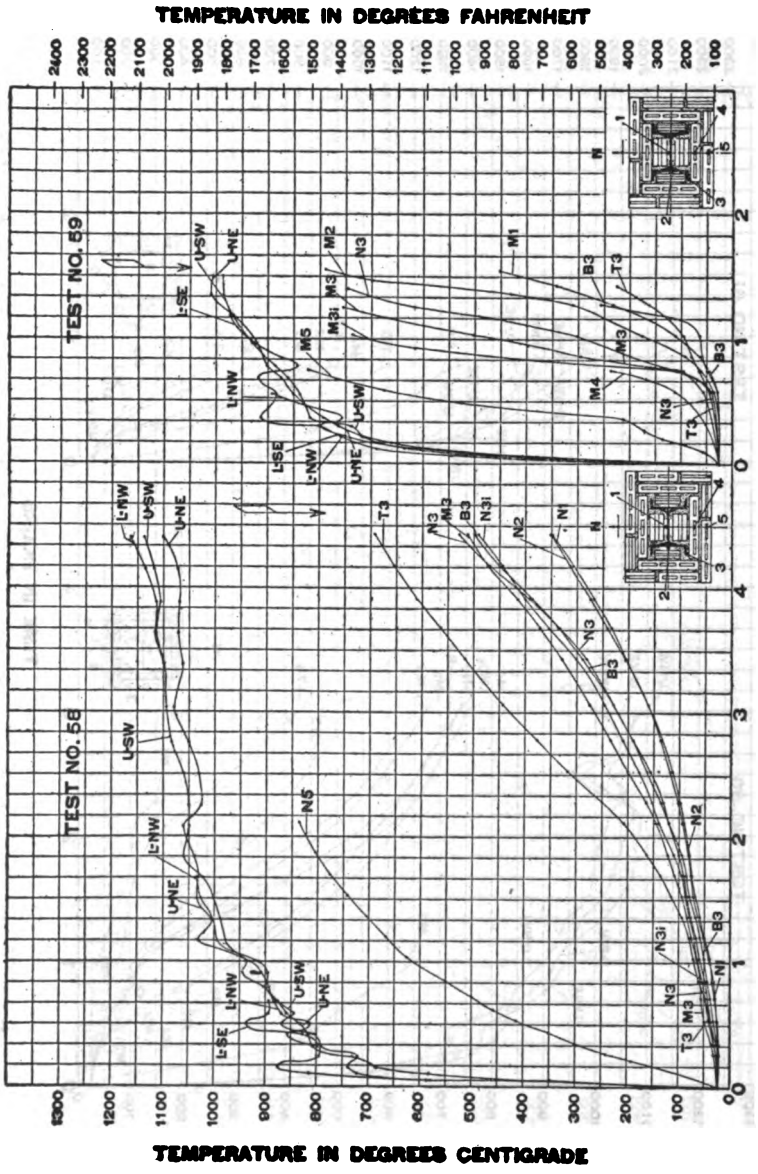

FIG. 124.—*Furnace and column temperatures, surface-clay tile protections, test Nos. 58 and 59*

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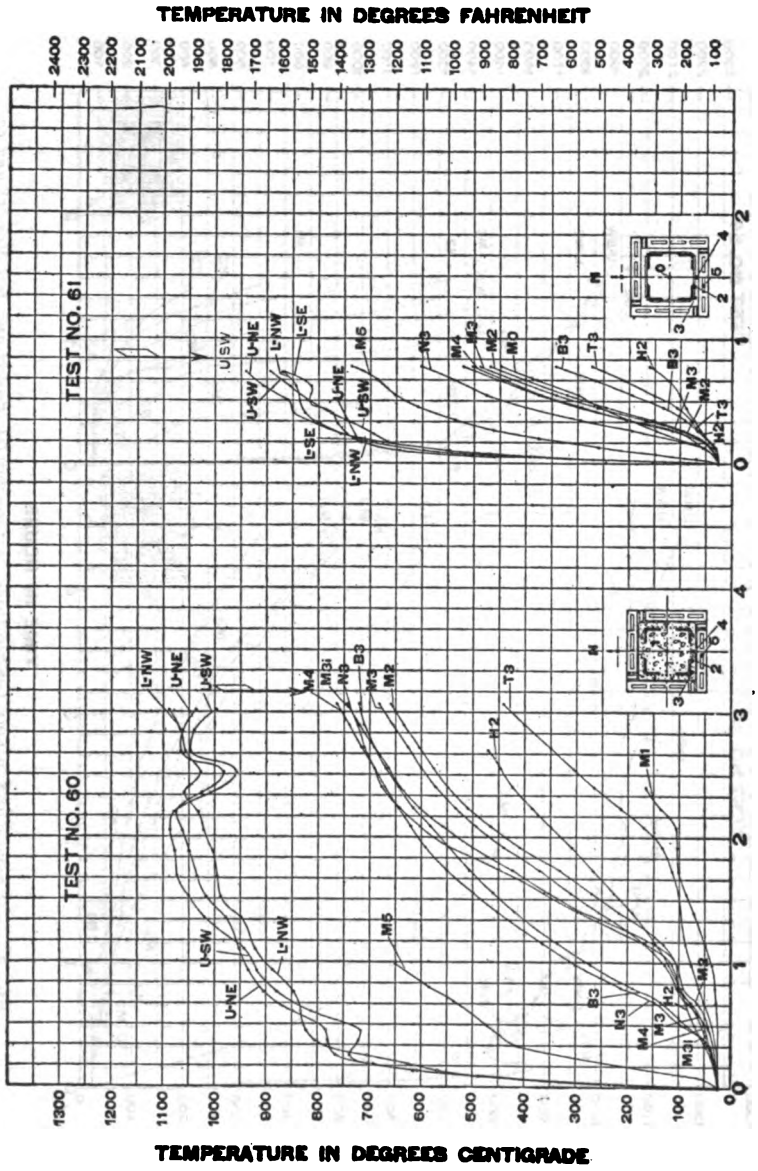

FIG. 125.—*Furnace and column temperatures, semifire-clay tile protections, test Nos. 60 and 61*

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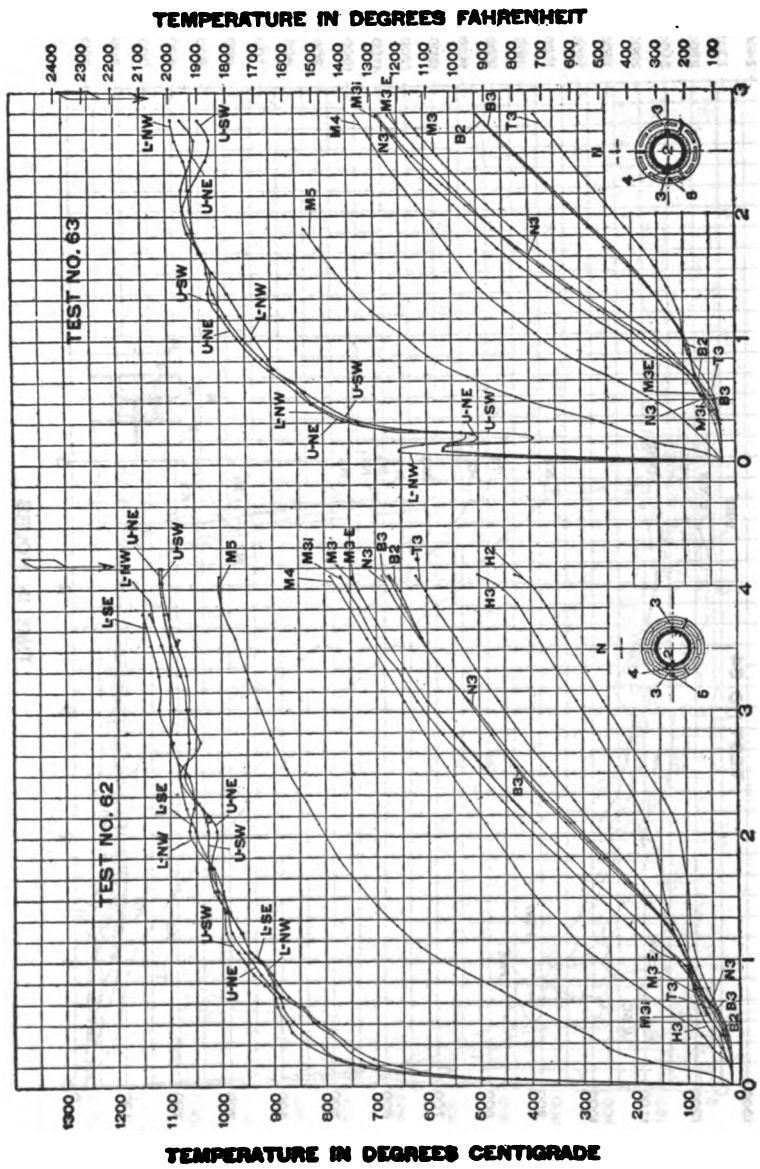

FIG. 126.—*Furnace and column temperatures, porous semifire-clay tile protections, test Nos. 62 and 63*

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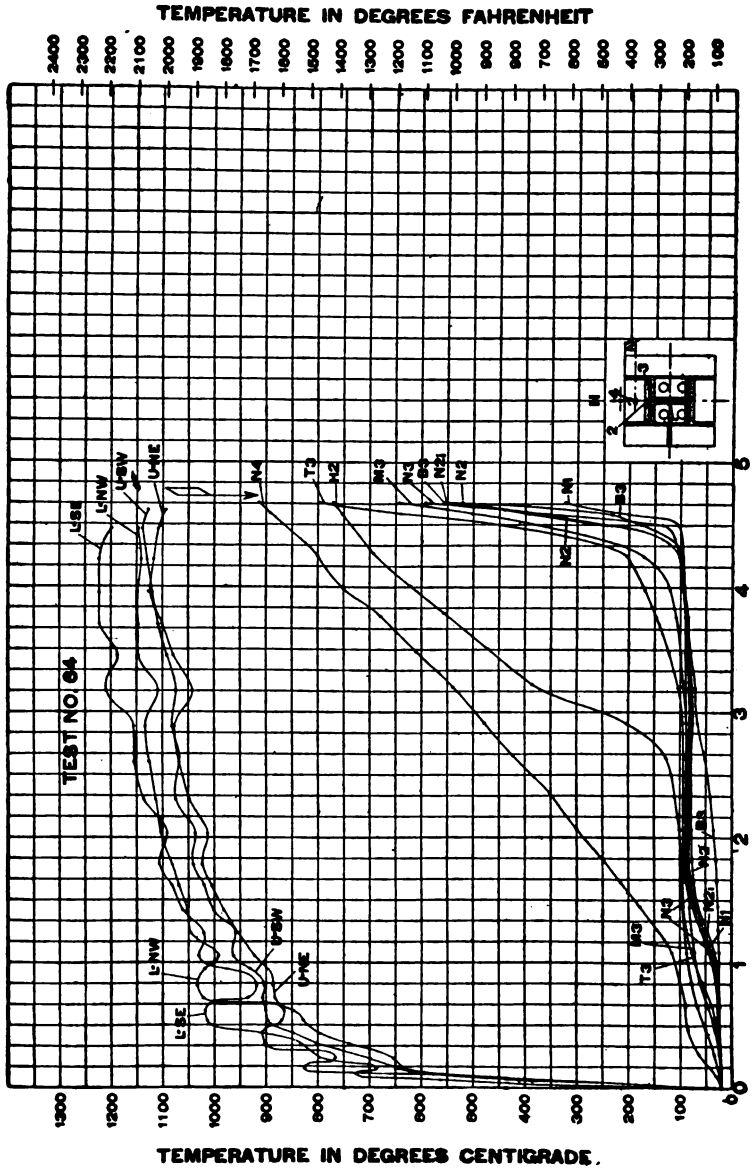

FIG. 127.—*Furnace and column temperatures, 4-inch gypsum-block protection, test No. 64*

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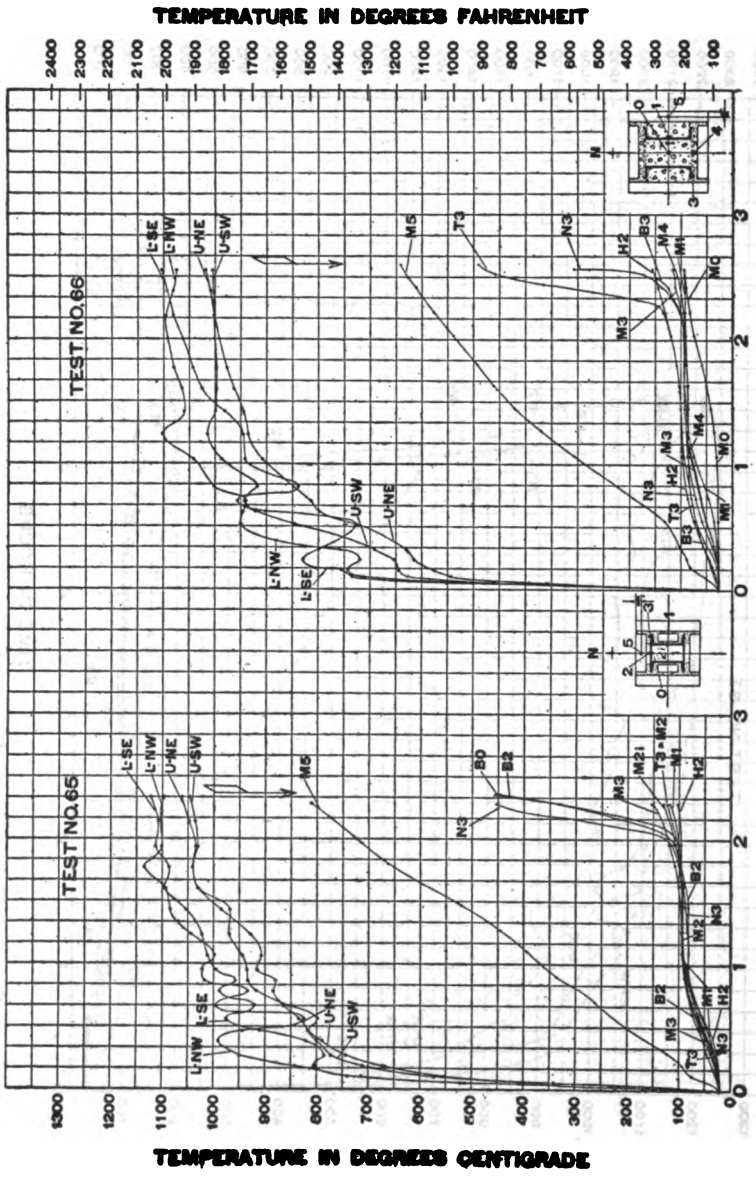

FIG. 128.—*Furnace and column temperatures, 2-inch gypsum-block protections, test Nos. 65 and 66*

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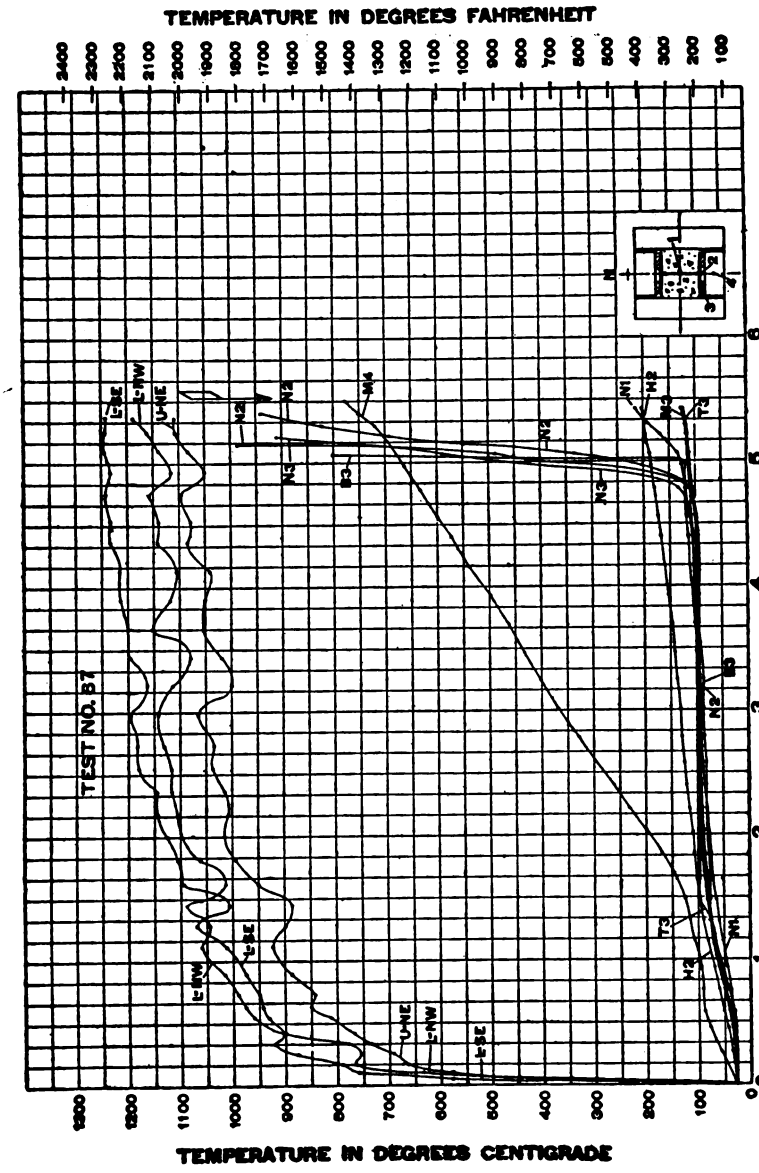

FIG. 129.—*Furnace and column temperatures, 4-inch gypsum-block protection, test No. 67*

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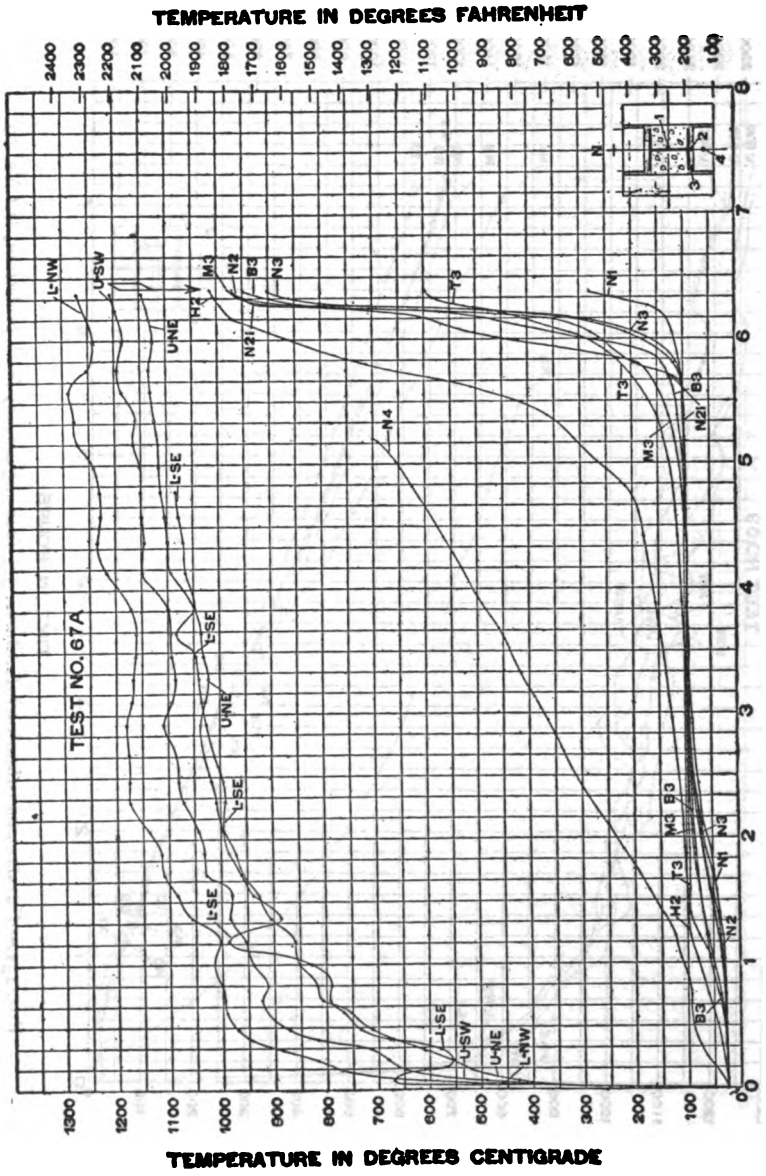

FIG. 130.—*Furnace and column temperatures, 4-inch gypsum-block protection, test No. 67A*

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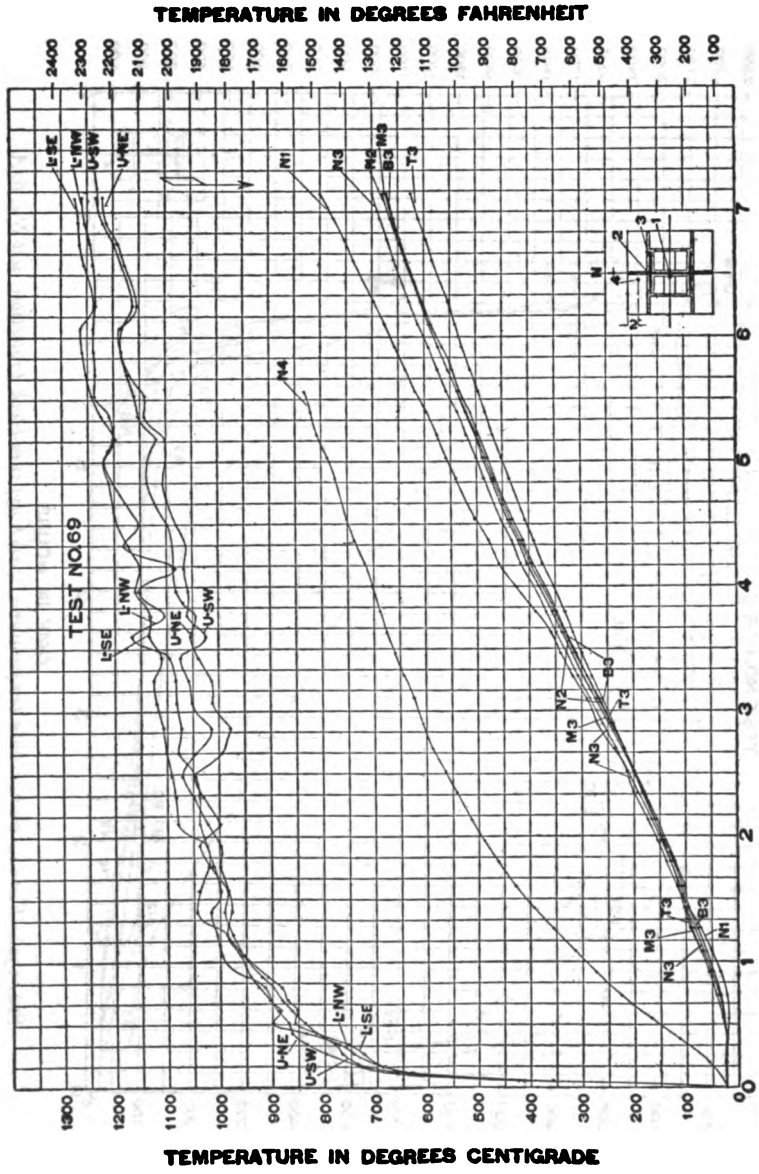

FIG. 131.—*Furnace and column temperatures, 3¼-inch surface-clay brick protection, test No. 69*

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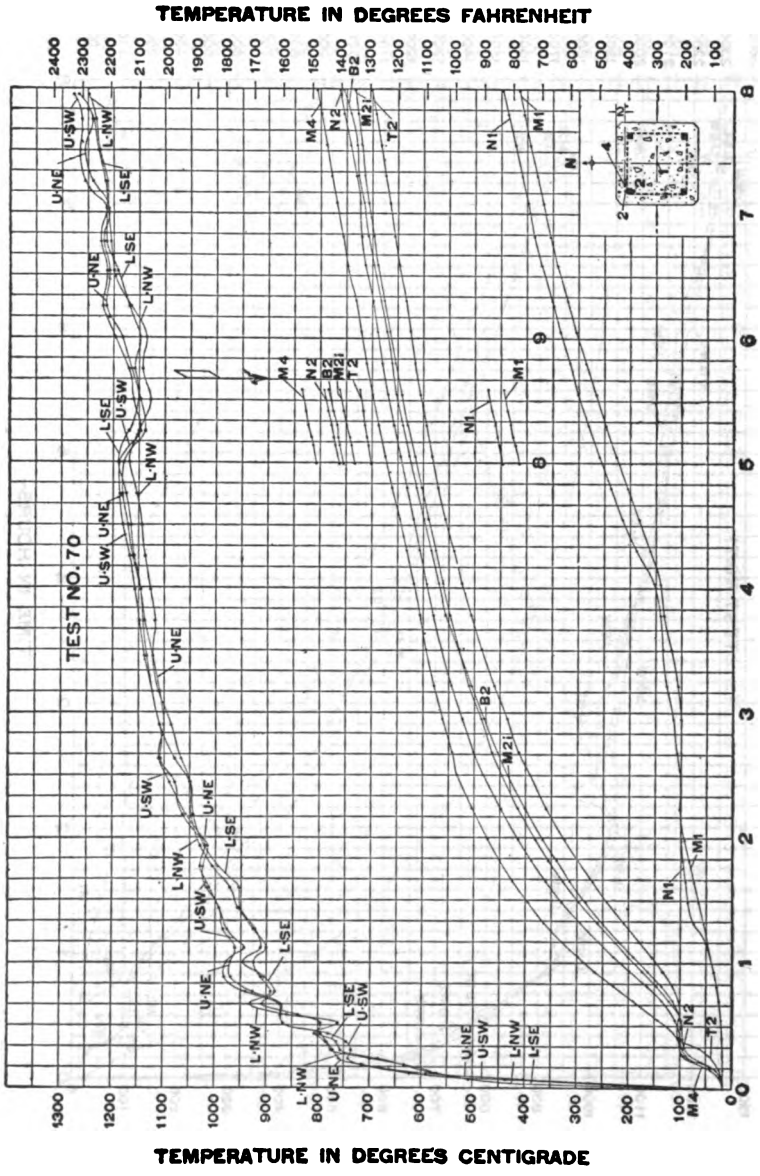

FIG. 132.—*Furnace and column temperatures, square, reinforced, limestone concrete column, test No. 70*

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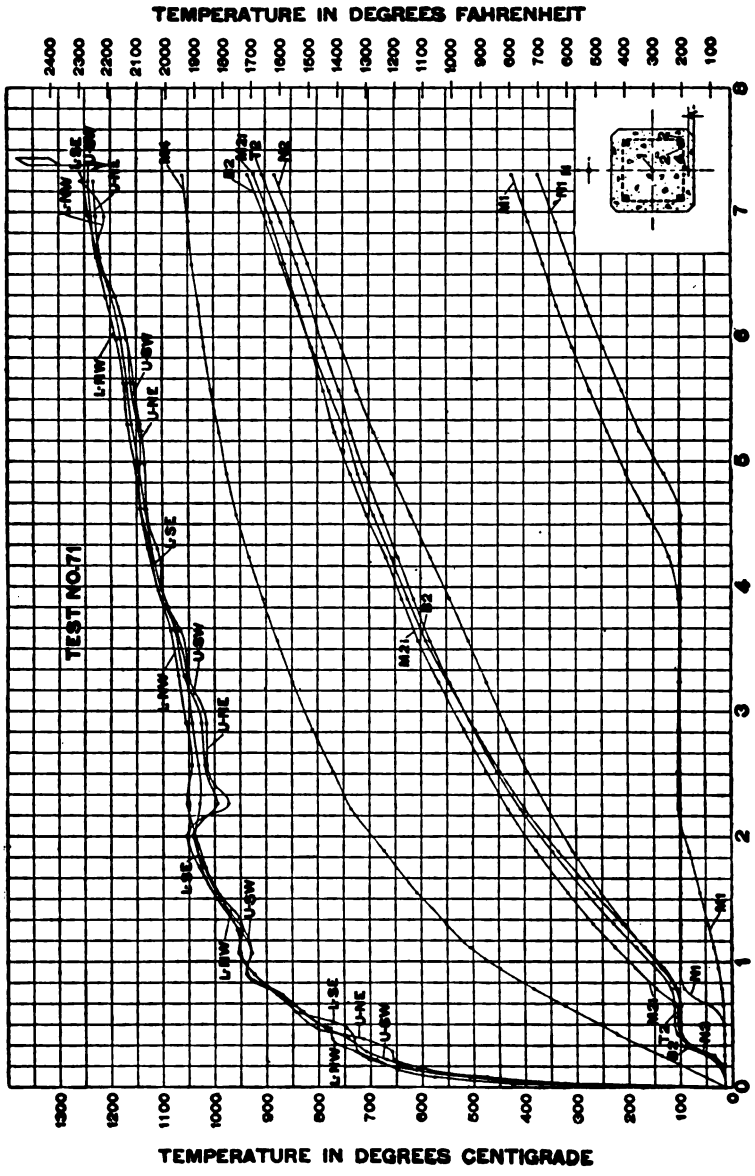

FIG. 133.—*Furnace and column temperatures, square, reinforced, trap-rock concrete column, test No. 71*

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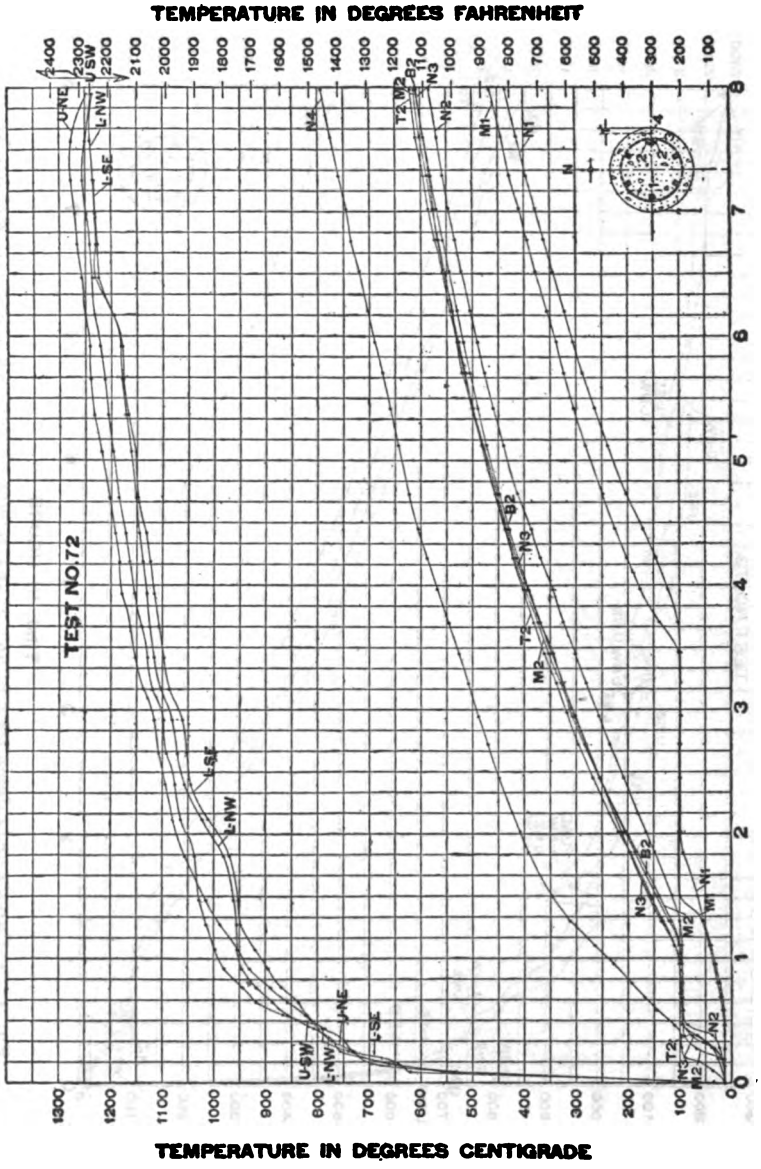

FIG. 134.—*Furnace and column temperatures, round, reinforced, limestone concrete columns, test No. 72*

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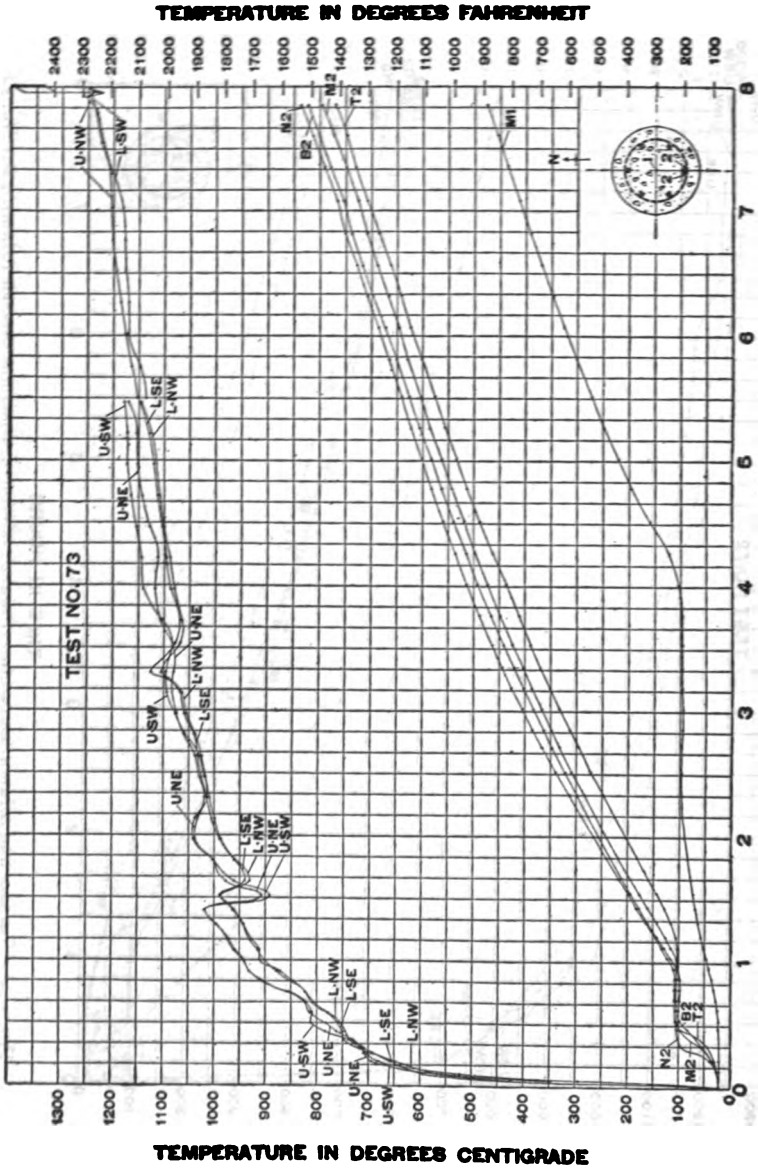

FIG. 135.—*Furnace and column temperatures, round, reinforced, trap-rock concrete column, test No. 73*

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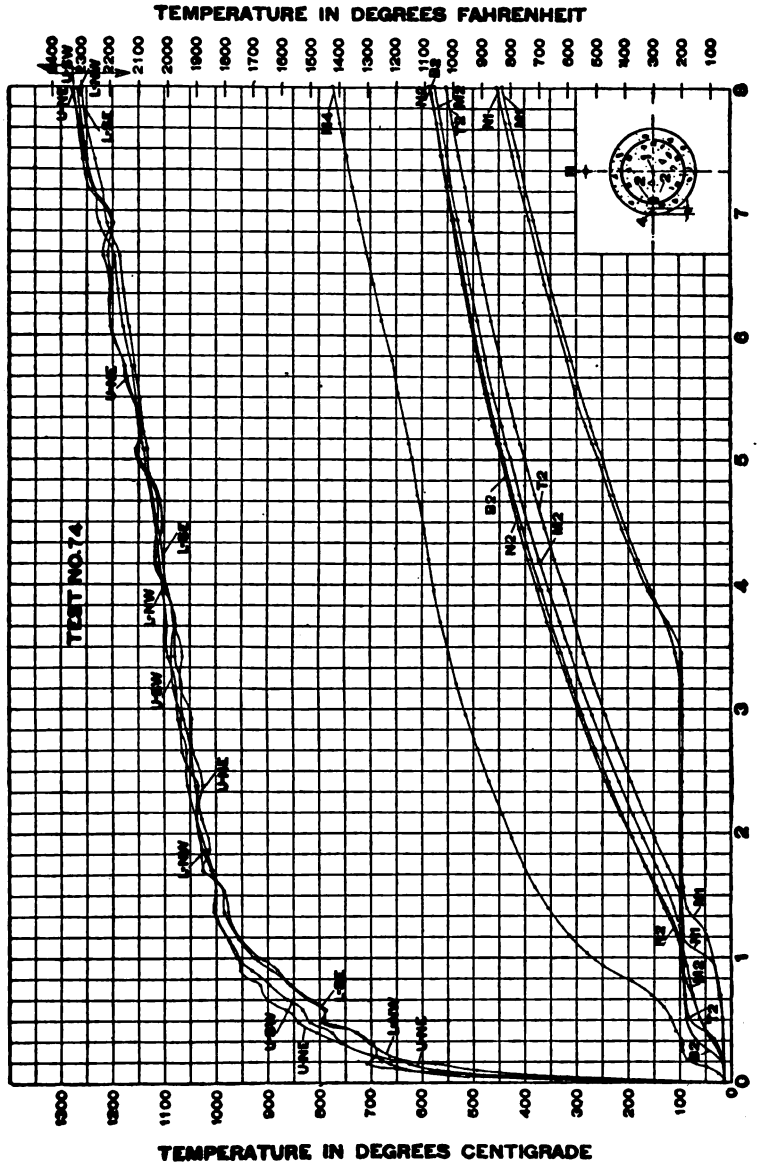

FIG. 136.—*Furnace and column temperatures, round, hooped, limestone concrete column, test No. 74*

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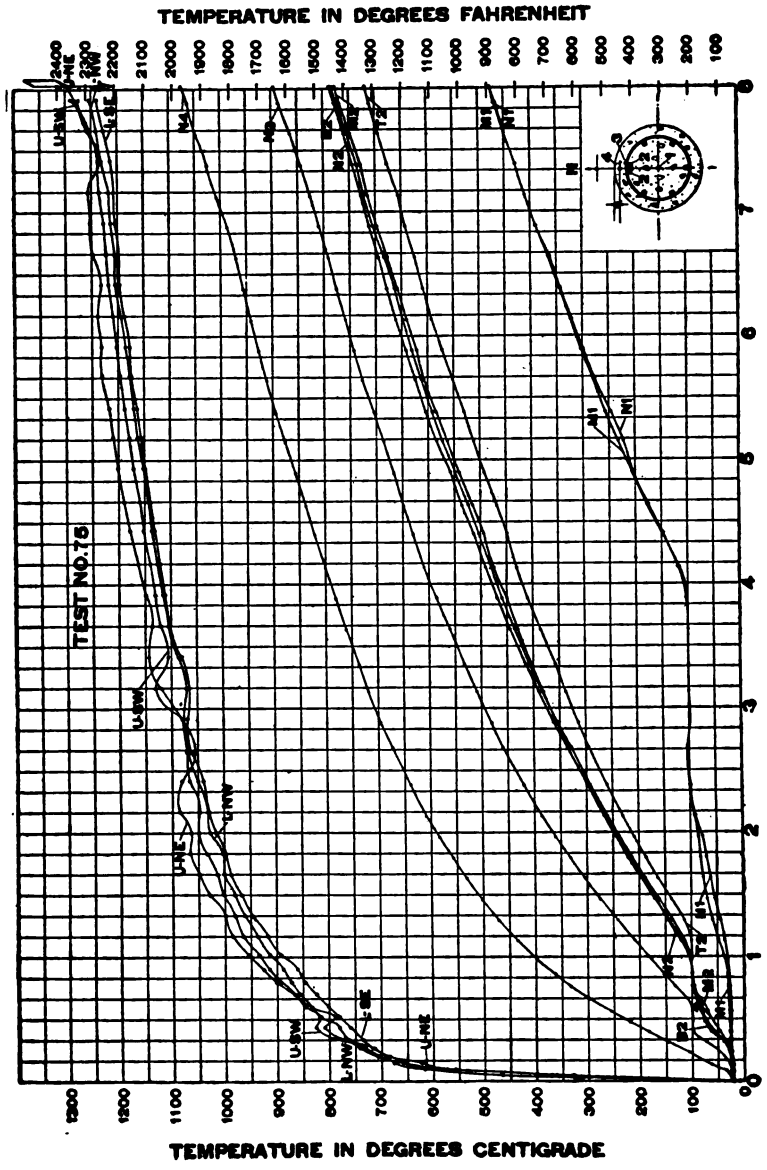

FIG. 137.—*Furnace and column temperatures, round, hooped, trap-rock concrete column, test No. 75*

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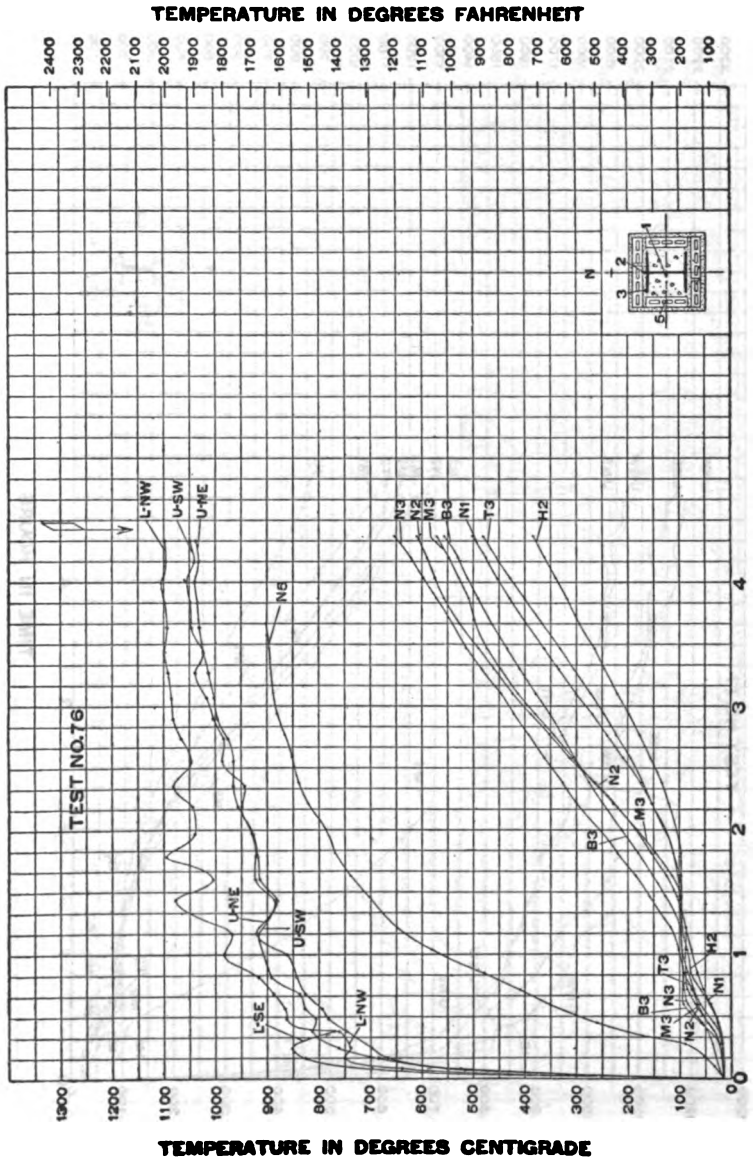

FIG. 138.—*Furnace and column temperatures, plastered clay-tile protection, test No. 76*

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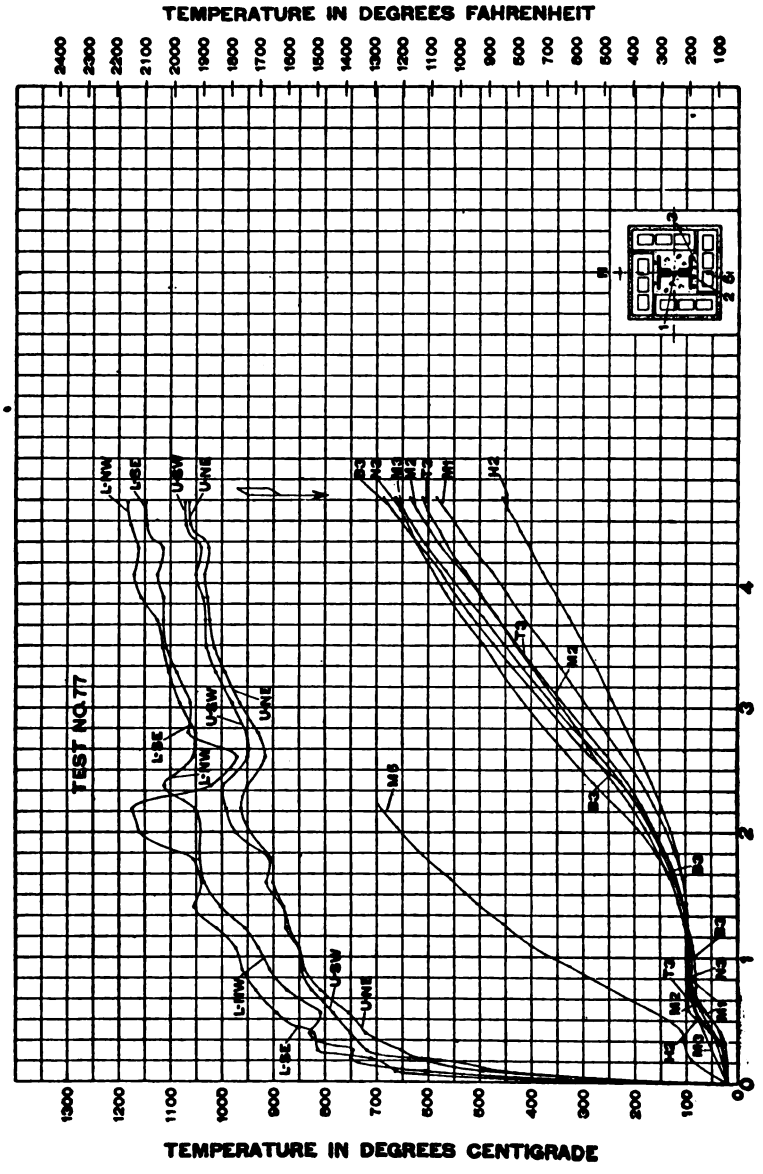

FIG. 139.—*Furnace and column temperatures, plastered clay-tile protection, test No. 77*

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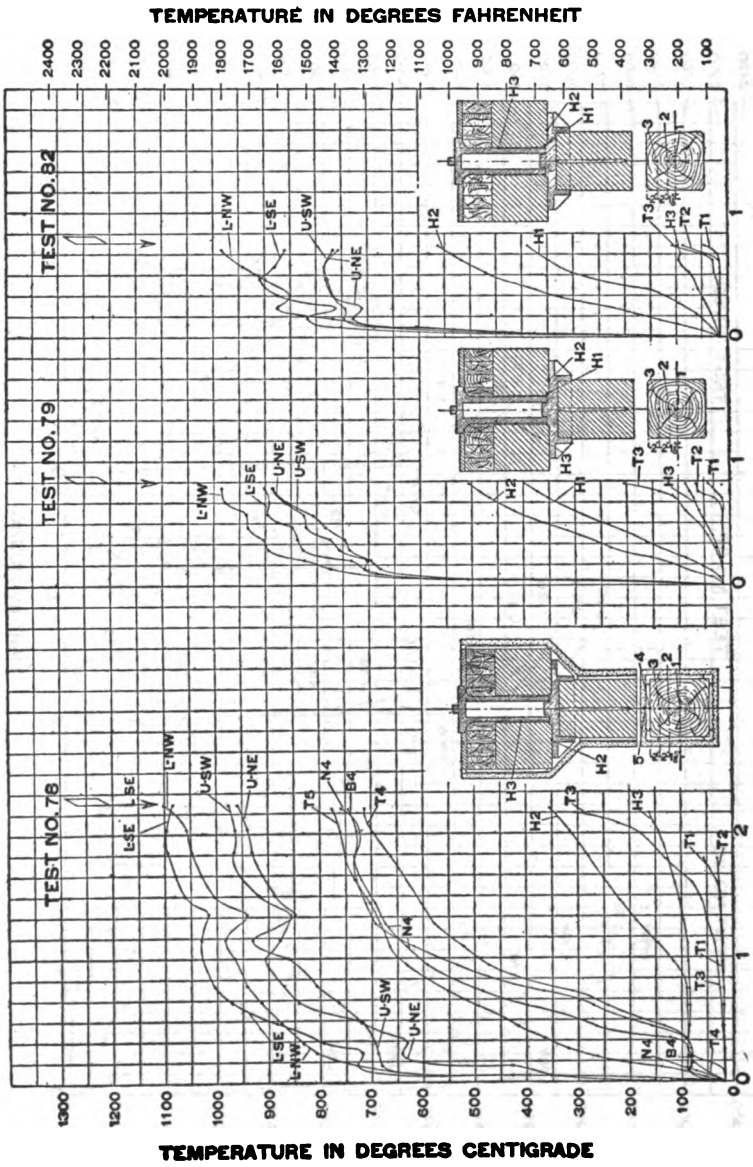

FIG. 140.—*Furnace and column temperatures, timber columns, cap and pintle bearings, test Nos. 78, 79, and 82*

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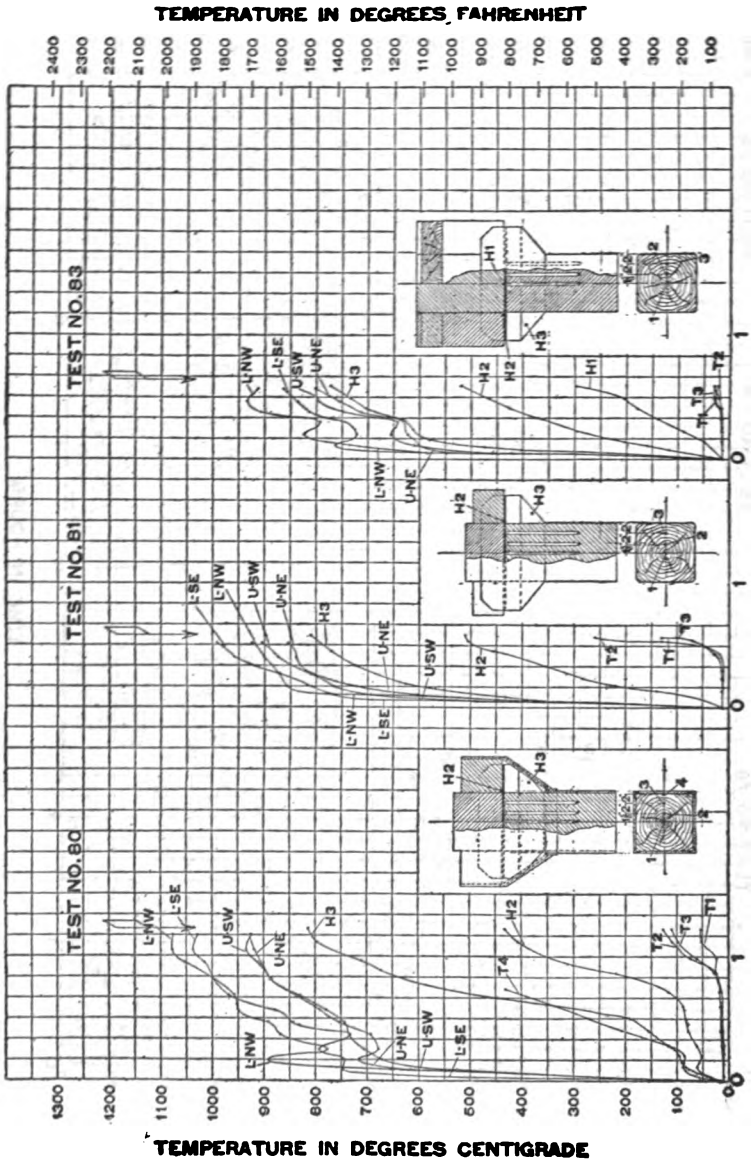

FIG. 141.—*Furnace and column temperatures, timber columns, steel-plate cap bearings, test Nos. 80, 81, and 83*

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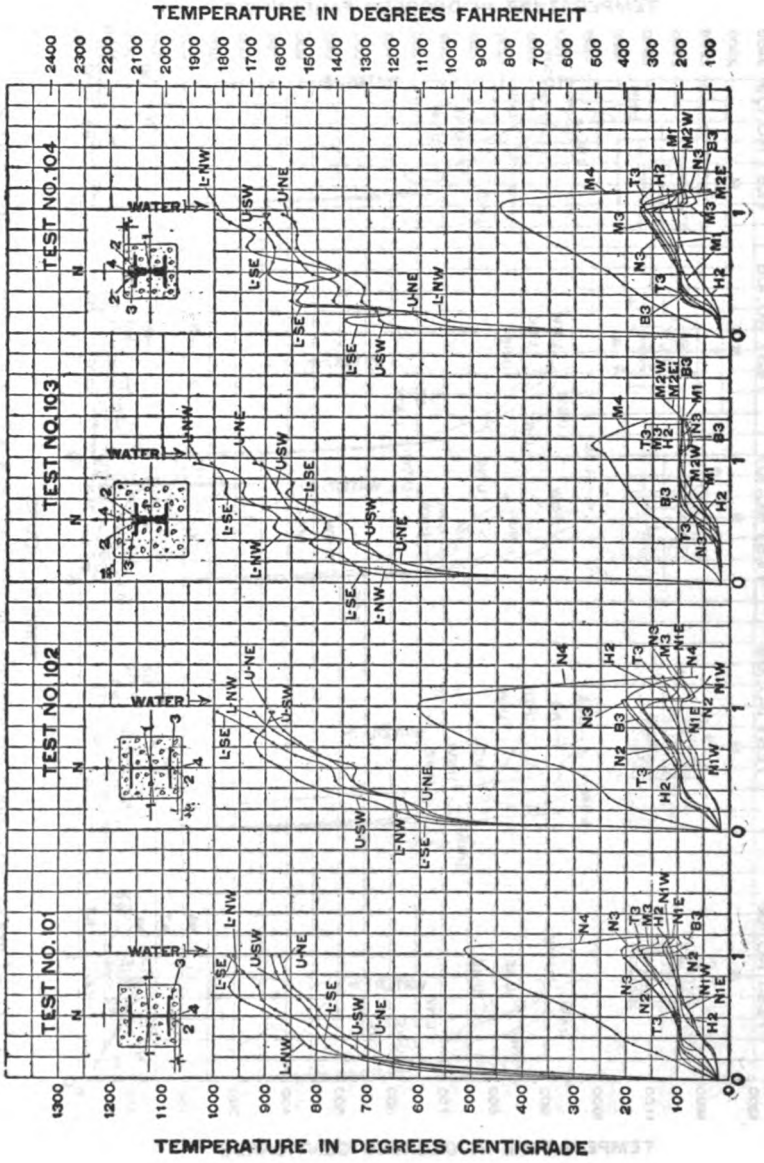

FIG. 142.—*Furnace and column temperatures, concrete protections, fire and water tests, Nos. 101 to 104*

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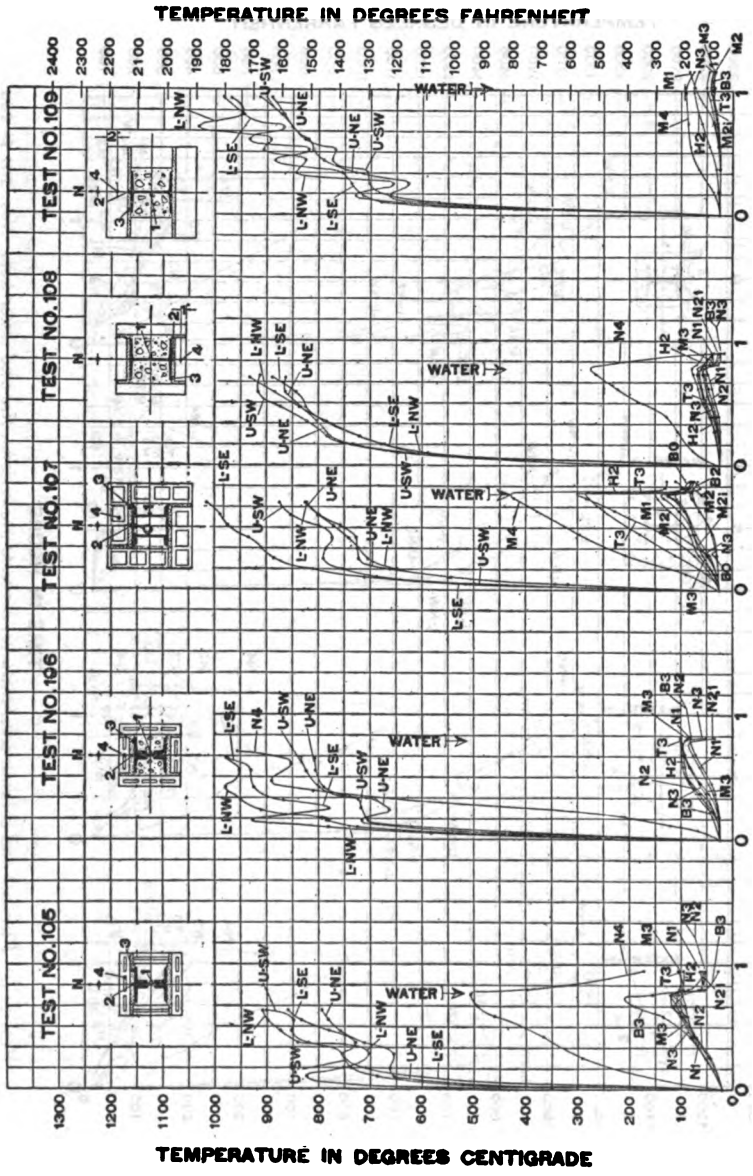

FIG. 143.—*Furnace and column temperatures, clay-tile and gypsum-block protections, fire and water tests, Nos. 105 to 109*

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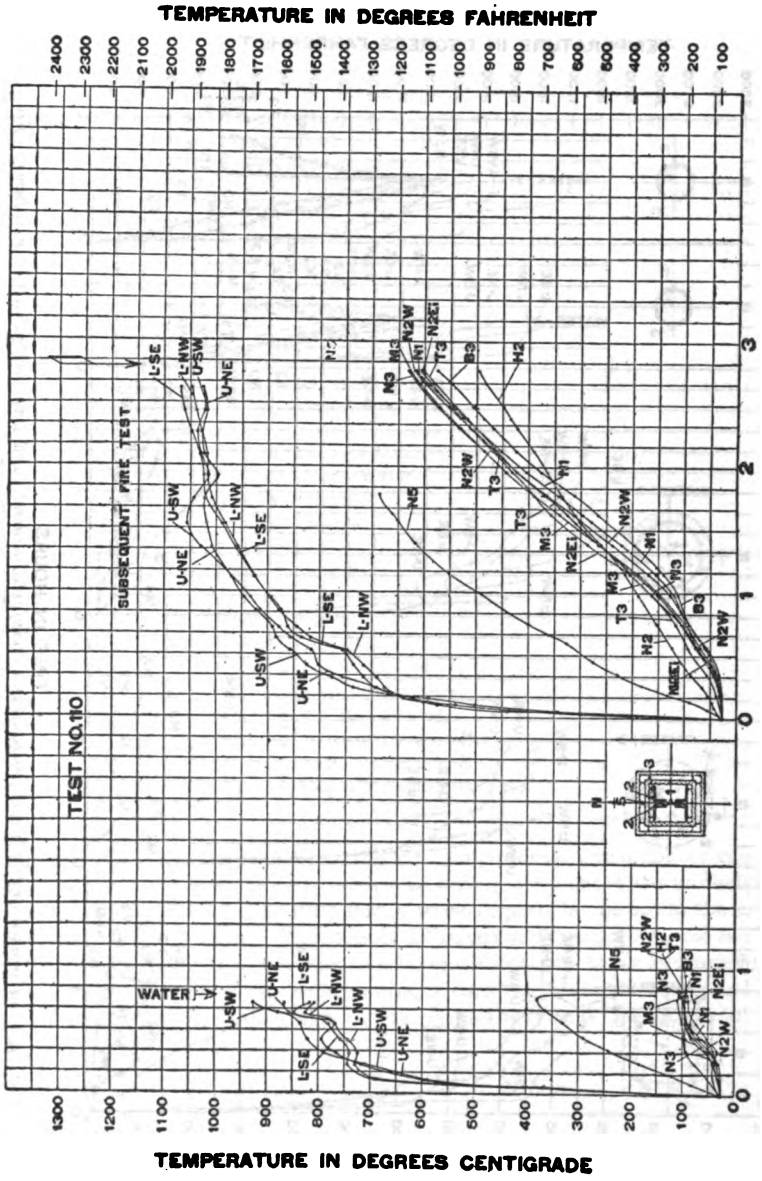

FIG. 144.—*Furnace and column temperatures, plaster on metal-lath protection, fire and water test, No. 110*

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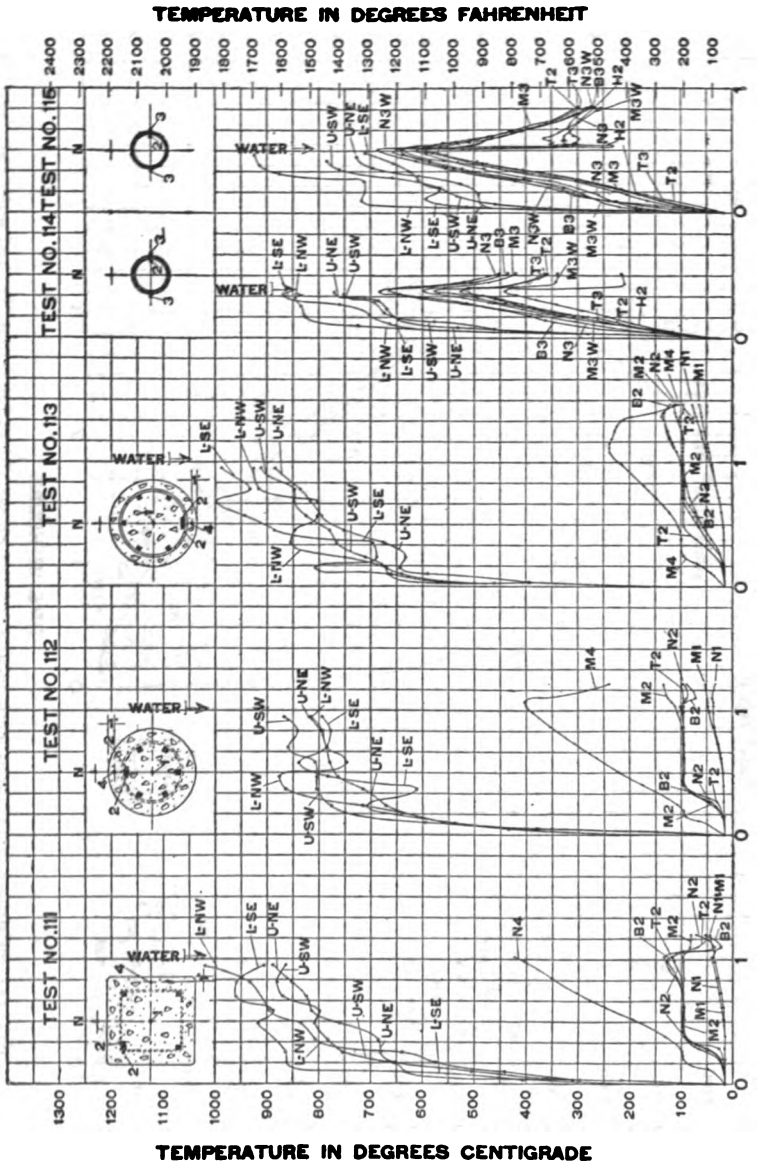

FIG. 145.—*Furnace and column temperatures, reinforced-concrete and unprotected cast-iron columns, fire and water tests, Nos. 111 to 115*

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APPENDIX C.—Deformation and Average Temperature Curves

Fig. No.	Test No.	Columns and protections	Page
146	1 to 5	Unprotected structural steel	306
147	9 to 11	Unprotected cast iron	309
147	12	Unprotected pipe	309
148	23, 24	Double layer plaster on metal lath	310
149	25, 26, 27	Single layer plaster on metal lath	311
150	28A	2-inch limestone concrete	312
151	29, 36	2-inch trap-rock concrete	313
152	31, 43	2-inch sandstone concrete	314
153	32, 32A	2-inch cinder concrete	315
154	33	4-inch limestone concrete	316
155	34	4-inch granite concrete	317
156	35	4-inch limestone concrete	318
151	36	2-inch trap-rock concrete	313
157	37	4-inch trap-rock concrete	319
158	38	2-inch Joliet gravel concrete	320
159	40	2-inch trap-rock concrete	321
160	41	4-inch limestone concrete	322
161	42	....do	322
152	43	2-inch sandstone concrete	314
158	47	2-inch cinder concrete	320
162	48, 49	Semifire-clay tile	324
163	50A, 51A	Surface-clay tile	325
163	52, 53	Shale tile	325
164	54, 55	Semifire-clay tile	325
165	56	....do	327
165	58	Surface-clay tile	327
166	60	Semifire-clay tile	325
167	62, 63	Porous semifire-clay tile	330
166	68	2 1/4-inch surface-clay brick	328
168	69	3 3/4-inch surface-clay brick	330
169	71	Square, reinforced, trap-rock concrete	331
170	72	Round, reinforced, limestone concrete	332
171	74	Round, hooped, limestone concrete	333

NOTE.—The unit deformation is the unit longitudinal expansion or compression observed over a 37-inch (94 cm) gage length immediately above the midheight of the columns, and is given on the diagrams in terms of parts in 100 000 to the same scale as centigrade temperature. Thus a unit deformation of 100 parts in 100 000 is 0.001. Deformations above the zero line denote expansion and below compression. The initial compression shown at the beginning of each test is due to the applied working load. Cf. Section VIII paragraph *2b*, Fig. 33, and Section X, paragraph *6a*.

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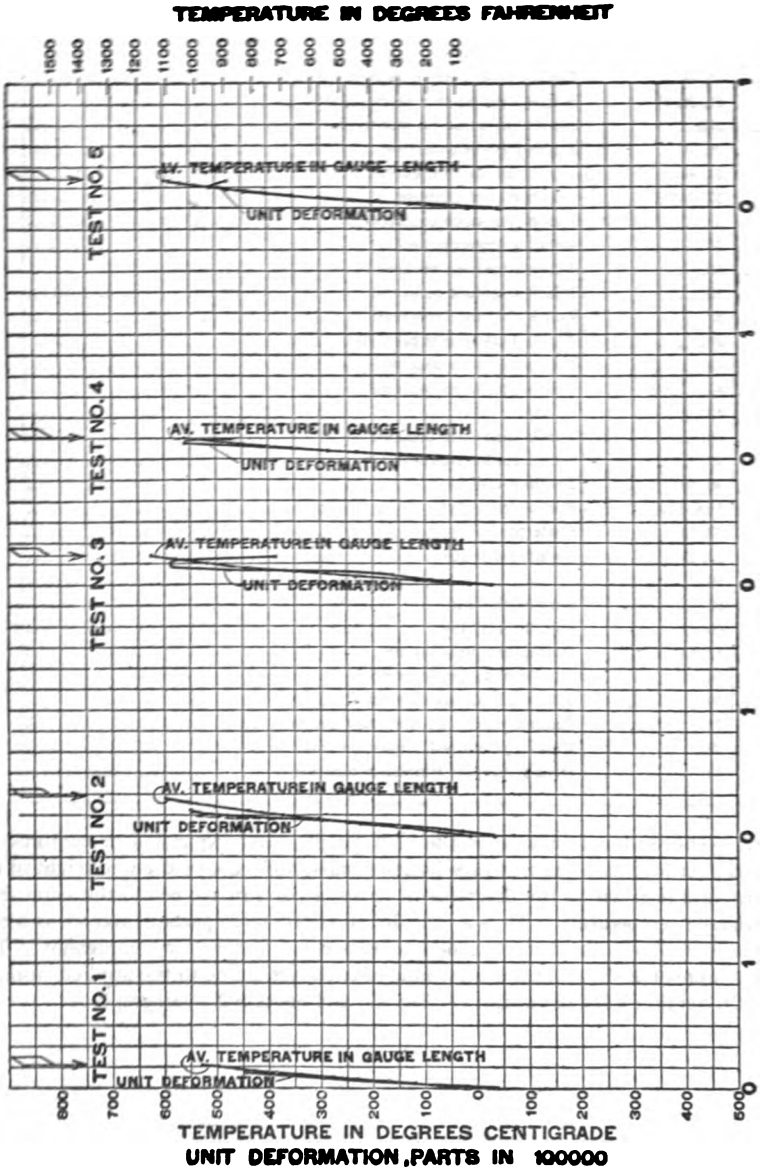
TEMPERATURE IN DEGREES FAHRENHEIT
1500 1400 1300 1200 1100 1000 900 800 700 600 500 400 300 200 100
TEST NO. 5
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 4
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 3
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 2
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 1
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TIME IN HOURS
800 700 600 500 400 300 200 100 0 100 200 300 400 500
TEMPERATURE IN DEGREES CENTIGRADE
UNIT DEFORMATION, PARTS IN 100000

FIG. 146.—*Average temperature and deformation, unprotected structural-steel columns, test Nos. 1 to 5*

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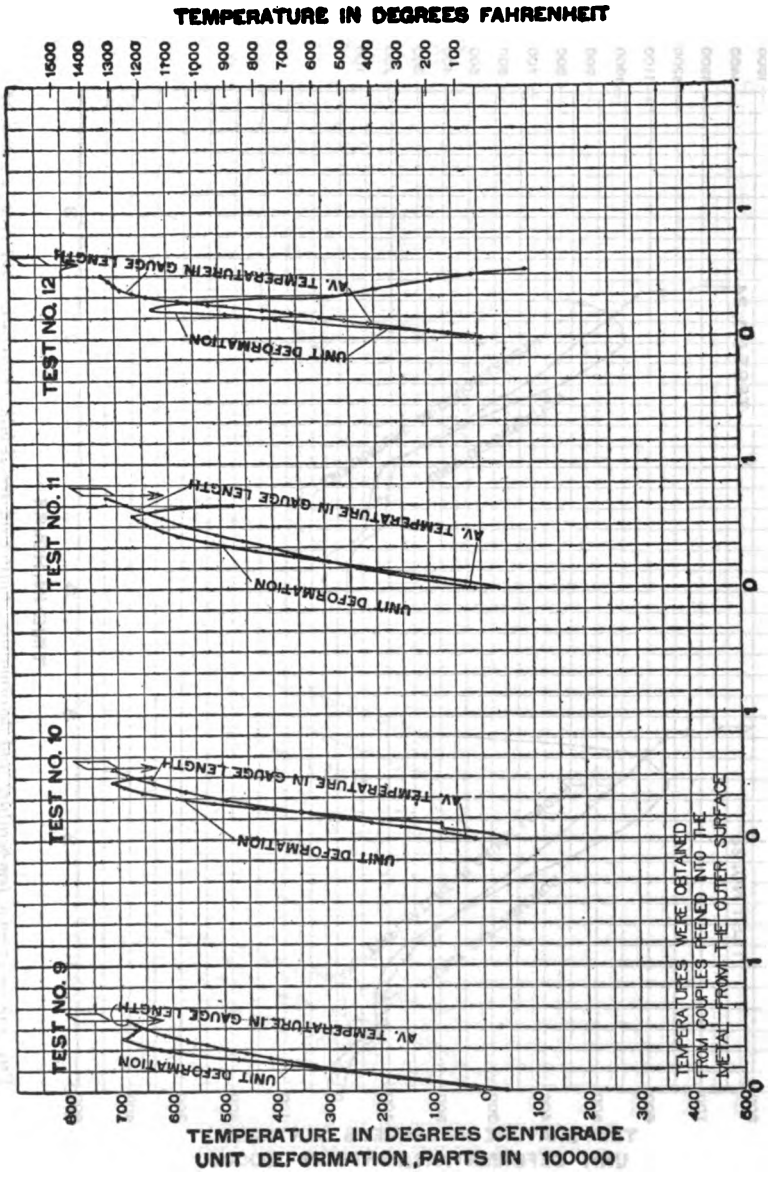
TEMPERATURE IN DEGREES FAHRENHEIT
1500 1400 1300 1200 1100 1000 900 800 700 600 500 400 300 200 100
TEST NO. 12
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 11
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 10
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 9
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TIME IN HOURS
TEMPERATURES WERE OBTAINED FROM COUPLES PEENED INTO THE METAL FROM THE OUTER SURFACE
TEMPERATURE IN DEGREES CENTIGRADE
UNIT DEFORMATION, PARTS IN 100000

FIG. 147.—*Average temperature and deformation, unprotected cast-iron and pipe columns, test Nos. 9 to 12*

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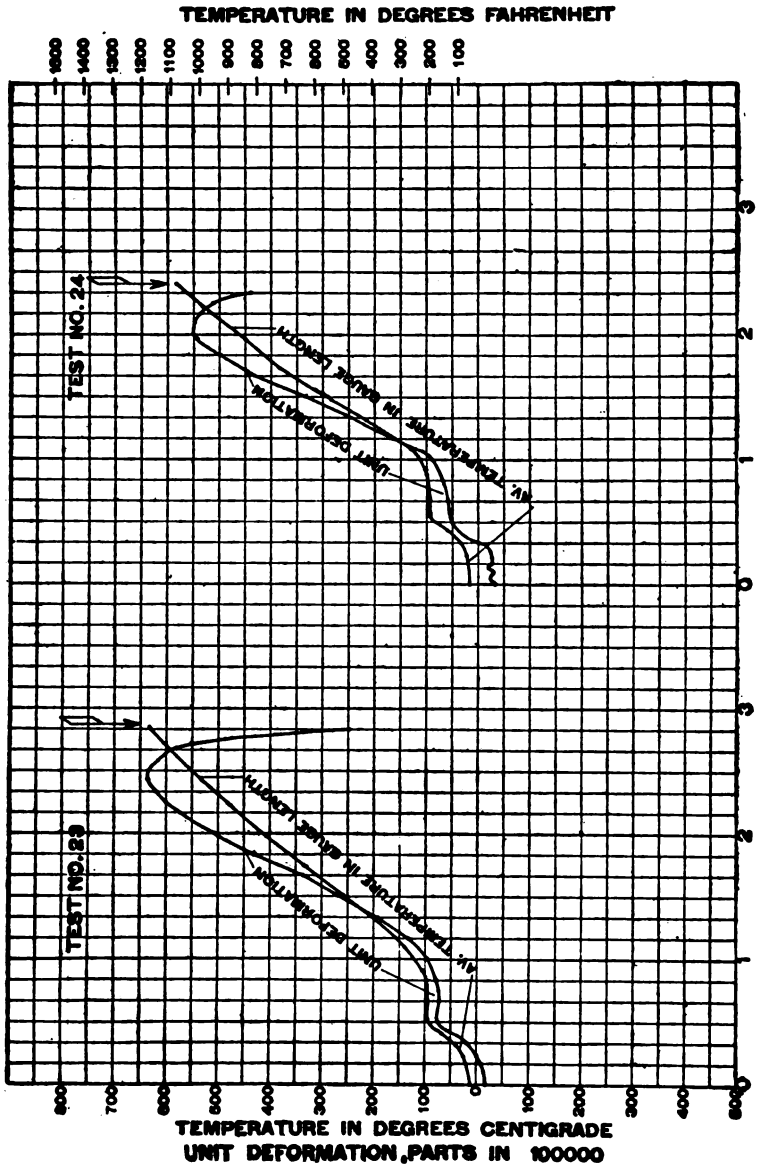

FIG. 148.—*Average temperature and deformation, plaster on metal-lath protections, test Nos. 23 and 24*

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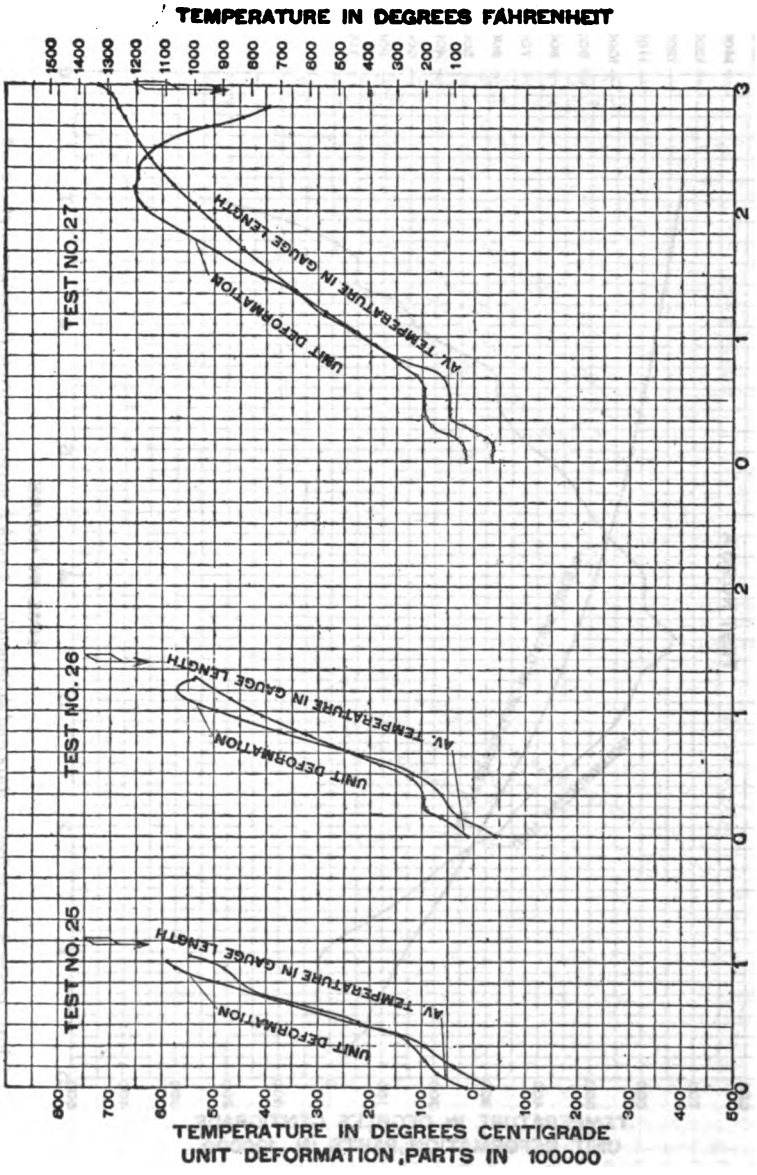
TEMPERATURE IN DEGREES FAHRENHEIT
1500 1400 1300 1200 1100 1000 900 800 700 600 500 400 300 200 100
TEST NO. 27
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 26
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 25
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
3 2 1 0 1 2 3
TIME IN HOURS
800 700 600 500 400 300 200 100 0 100 200 300 400 500
TEMPERATURE IN DEGREES CENTIGRADE
UNIT DEFORMATION, PARTS IN 100000

FIG. 149.—*Average temperature and deformation, plaster on metal-lath protections, test Nos. 25, 26, and 27*

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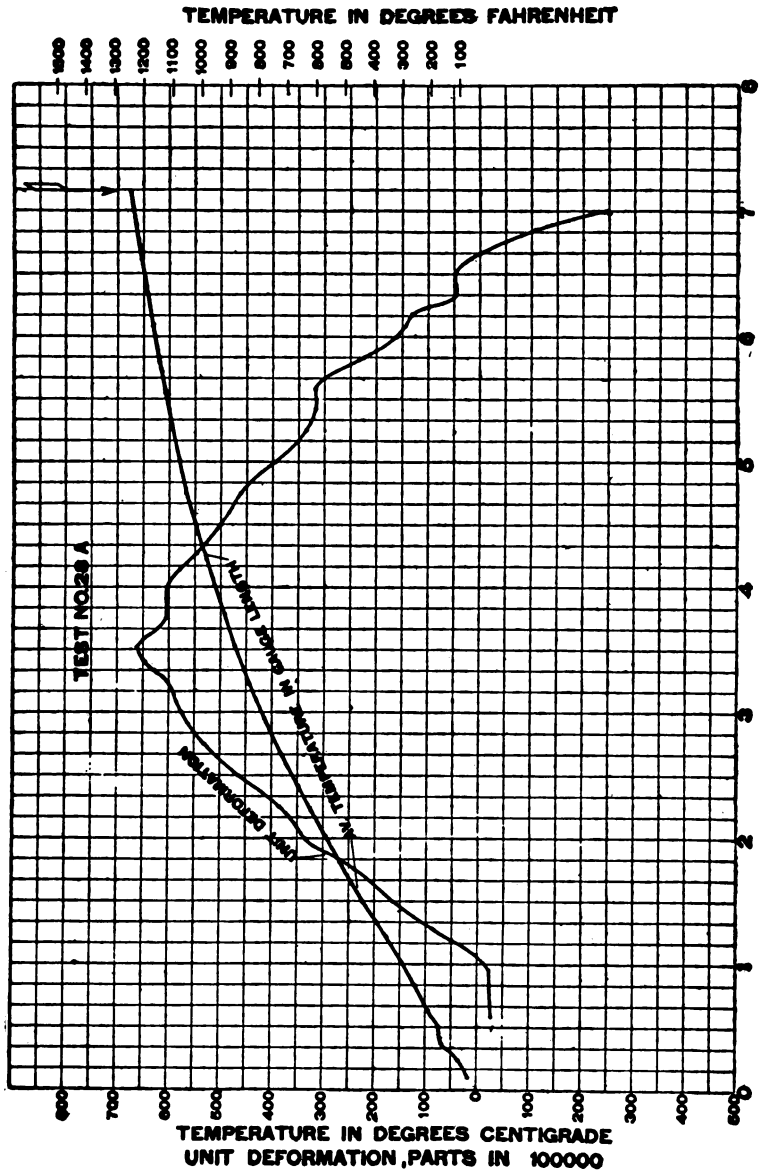
TEMPERATURE IN DEGREES FAHRENHEIT
1500 1400 1300 1200 1100 1000 900 800 700 600 500 400 300 200 100
TEST NO.28 A
TIME IN HOURS
0 1 2 3 4 5 6
LINE OF TEMPERATURE IN CHIMNEY
LINE OF DEFORMATIONS
600 700 800 500 400 300 200 100 0 100 200 300 400 600
TEMPERATURE IN DEGREES CENTIGRADE
UNIT DEFORMATION, PARTS IN 100000

FIG. 150.—*Average temperature and deformation, 2-inch limestone concrete protection, test No. 28A*

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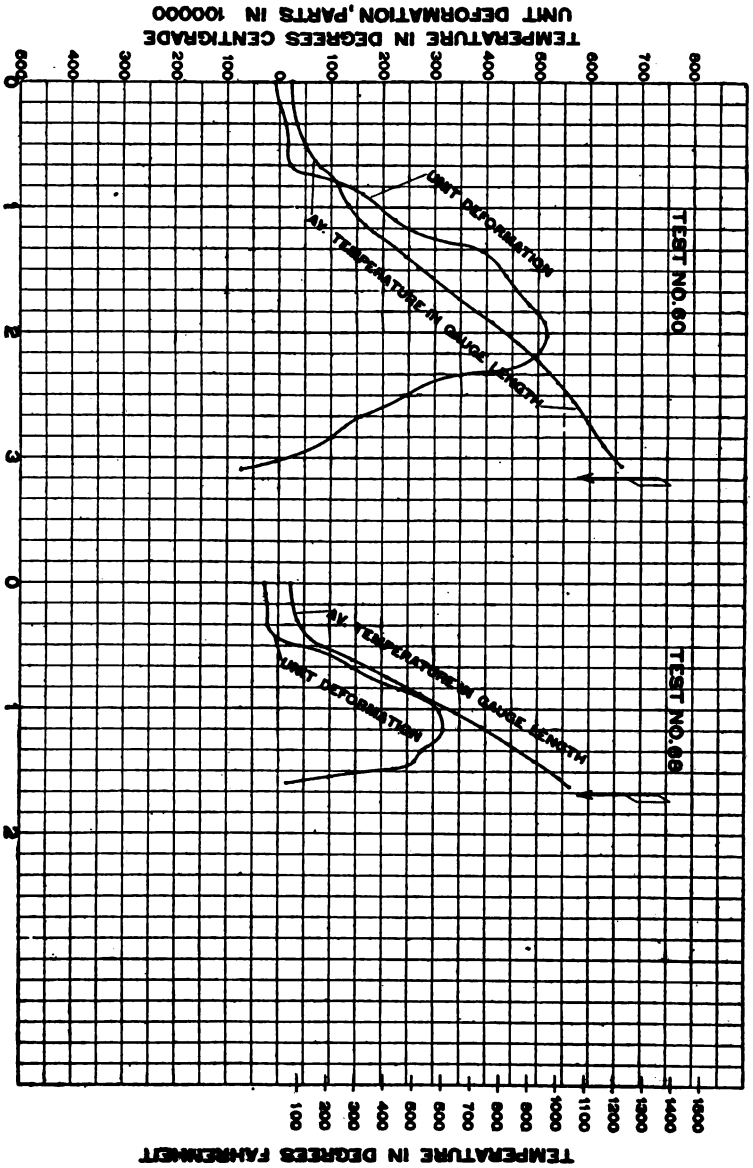

FIG. 166.—*Average temperature and deformation, semifire-clay tile protection, test No. 60; 2¼-inch surface-clay brick protection, test No. 68*

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FIG. 165.—*Average temperature and deformation, semifire-clay tile protection, test No. 56; surface-clay tile protection, test No. 58*

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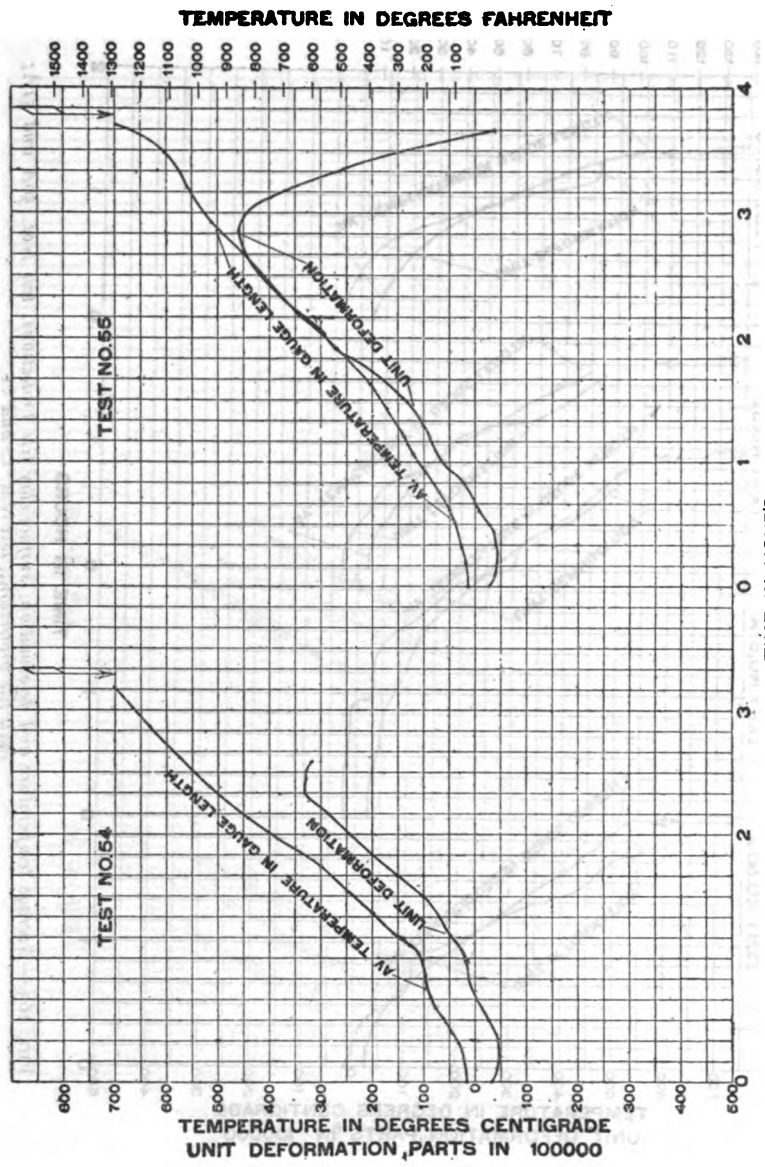

FIG. 164.—*Average temperature and deformation, semifire-clay tile protections, test Nos. 54 and 55*

FIG. 163.—*Average temperature and deformation, surface-clay tile protections, test Nos. 50A and 51A; shale tile protections, test Nos. 52 and 53*

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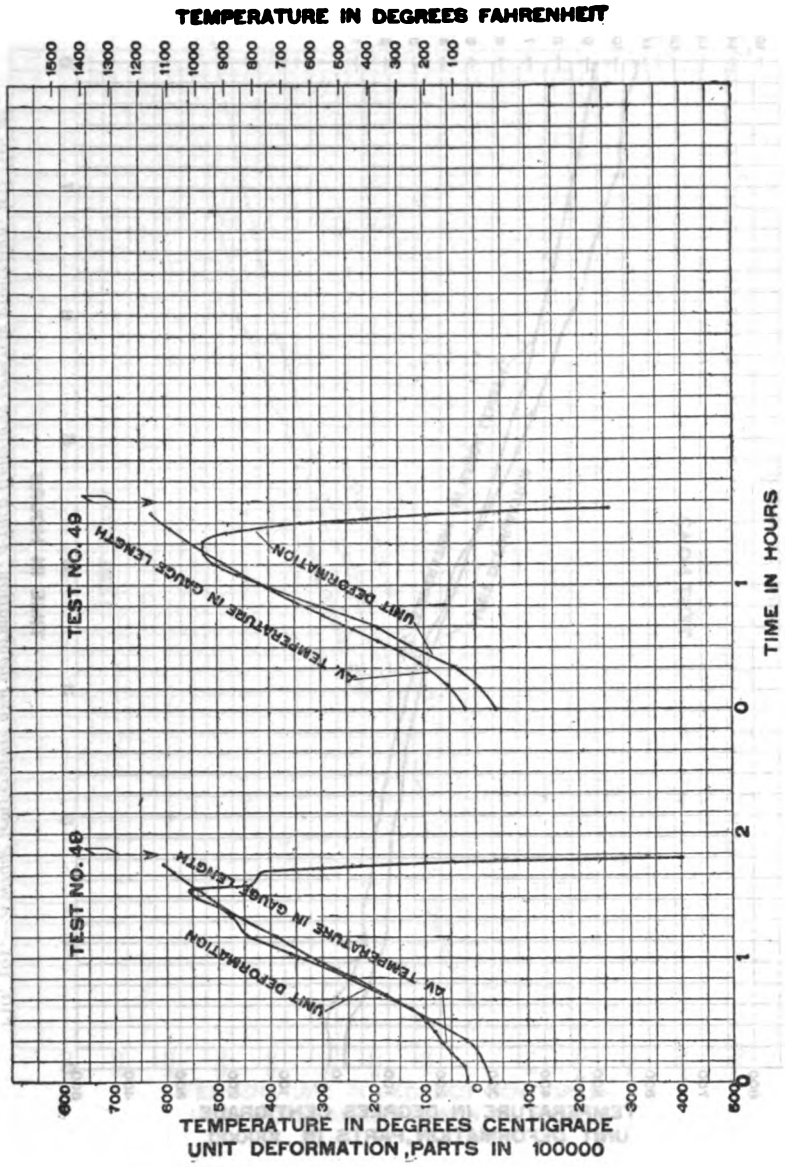
TEMPERATURE IN DEGREES FAHRENHEIT
1500 1400 1300 1200 1100 1000 900 800 700 600 500 400 300 200 100
TEST NO. 49
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TEST NO. 48
AV. TEMPERATURE IN GAUGE LENGTH
UNIT DEFORMATION
TIME IN HOURS
0 1 2 1 0
600 700 600 500 400 300 200 100 0 100 200 300 400 500
TEMPERATURE IN DEGREES CENTIGRADE
UNIT DEFORMATION, PARTS IN 100000

FIG. 162.—*Average temperature and deformation, semifire-clay tile protection, test Nos. 48 and 49*

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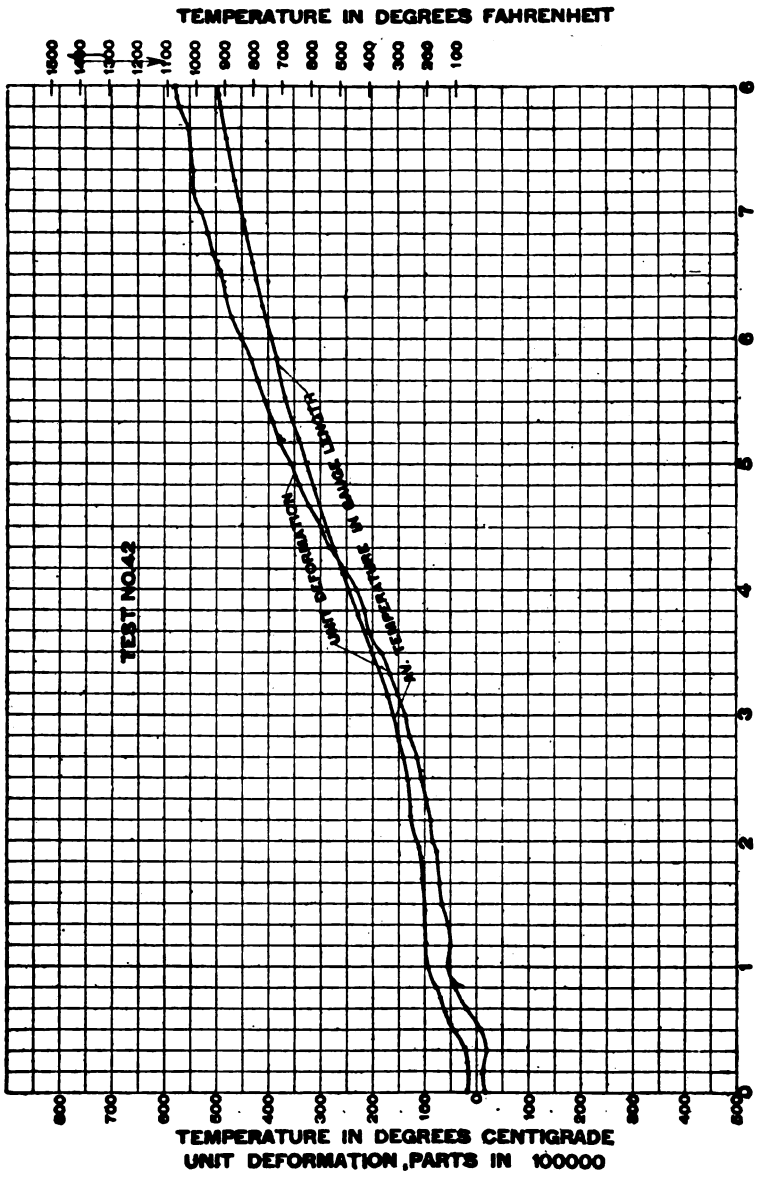

FIG. 161.—*Average temperature and deformation, 4-inch limestone concrete protection, test No. 42*

FIG. 160.—*Average temperature and deformation, 4-inch limestone concrete protection, test No. 41*

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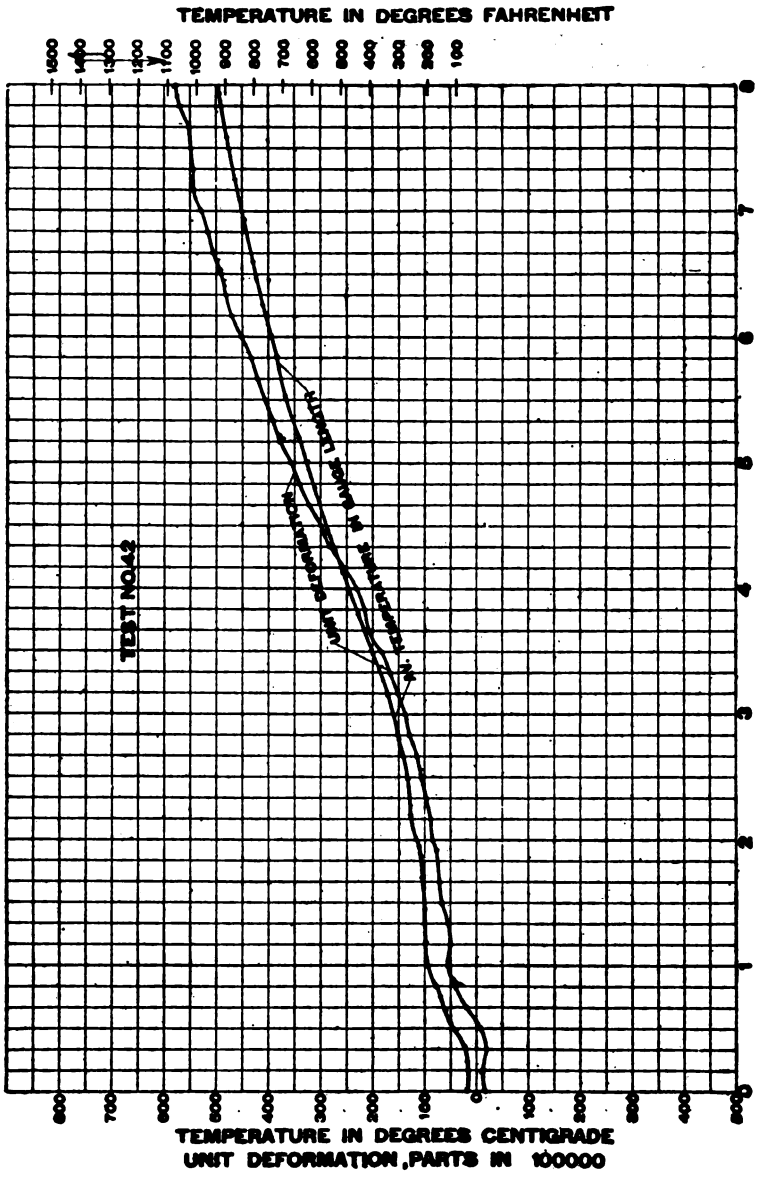

FIG. 161.—*Average temperature and deformation, 4-inch limestone concrete protection, test No. 42*

FIG. 160.—*Average temperature and deformation, 4-inch limestone concrete protection, test No. 41*

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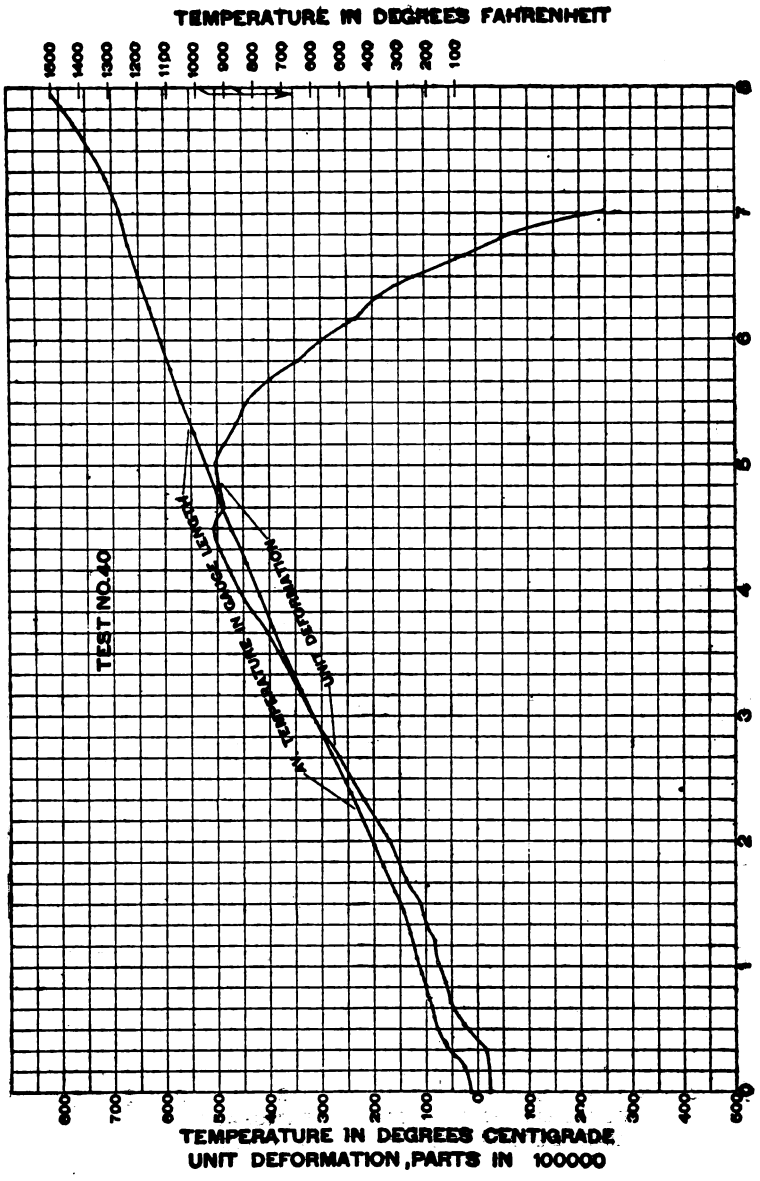
TEMPERATURE IN DEGREES FAHRENHEIT
1500 1400 1300 1200 1100 1000 900 800 700 600 500 400 300 200 100
TEST NO.40
AV. TEMPERATURE IN DEGREES FAHRENHEIT
UNIT DEFORMATION
TIME IN HOURS
0 1 2 3 4 5 6 7 8
600 700 800 500 400 300 200 100 0 100 200 300 400 500
TEMPERATURE IN DEGREES CENTIGRADE
UNIT DEFORMATION, PARTS IN 100000

FIG. 159.—*Average temperature and deformation, 2-inch trap-rock concrete protection, test No. 40*

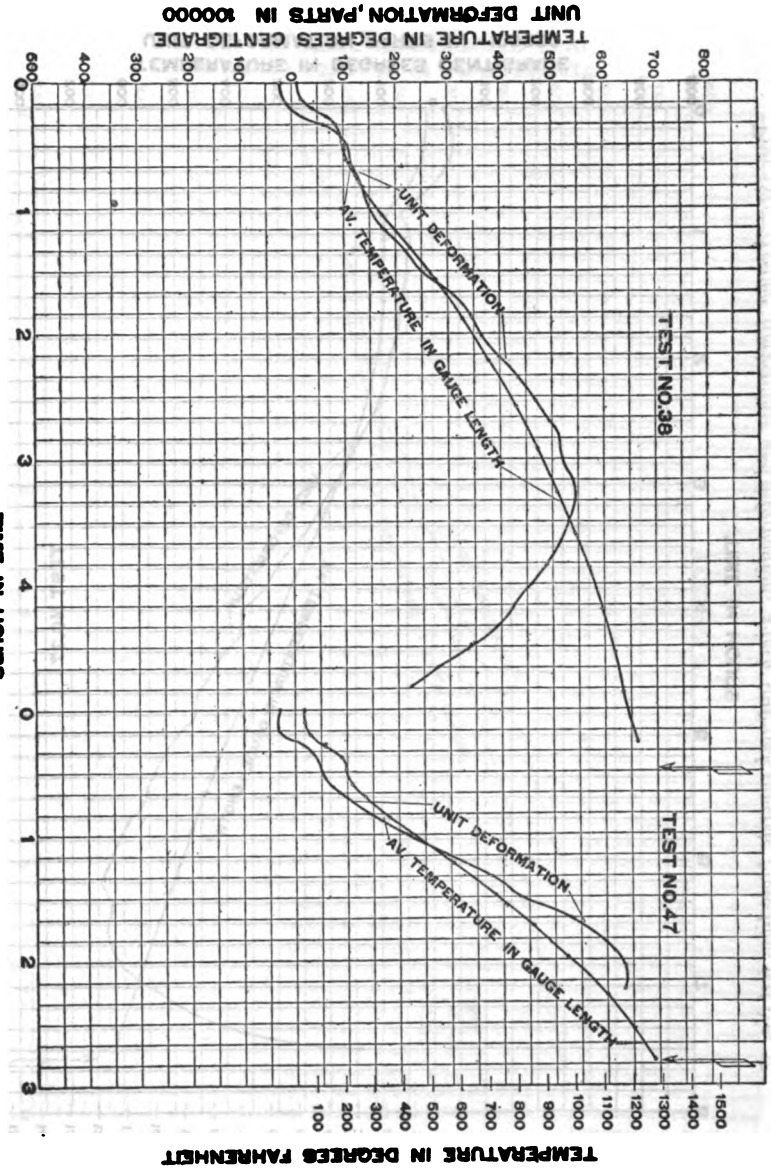

FIG. 158.—*Average temperature and deformation, 2-inch Joliet gravel concrete protection, test No. 38; 2-inch cinder concrete protection, test No. 47*

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FIG. 157.—*Average temperature and deformation, 4-inch trap-rock concrete protection, test No. 37*

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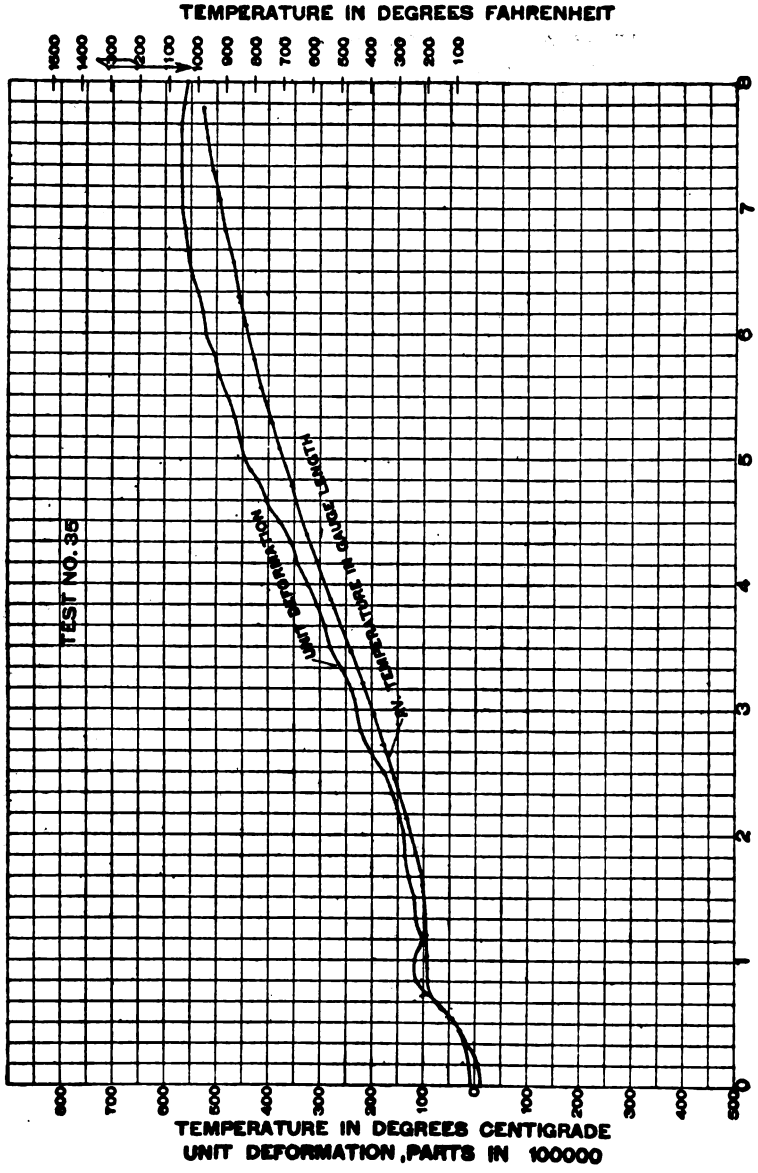
TEMPERATURE IN DEGREES FAHRENHEIT
1600 1400 1300 1200 1100 1000 900 800 700 600 500 400 300 200 100
TEST NO. 35
UNIT DEFORMATION
AV. TEMPERATURES IN CANAL LENGTH
TIME IN HOURS
0 1 2 3 4 5 6 7 8
800 700 600 500 400 300 200 100 0 100 200 300 400 500
TEMPERATURE IN DEGREES CENTIGRADE
UNIT DEFORMATION, PARTS IN 100000

FIG. 156.—*Average temperature and deformation, 4-inch limestone concrete protection, test No. 35*

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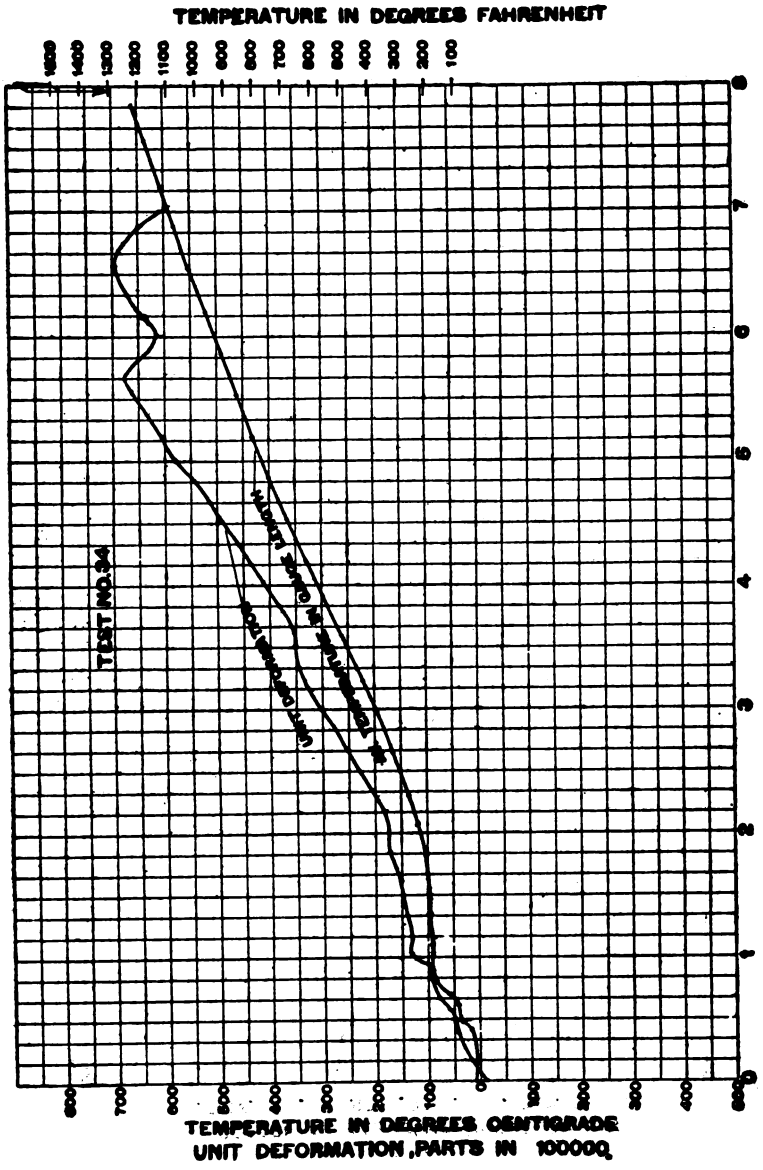

FIG. 155.—*Average temperature and deformation, 4-inch granite concrete protection, test No. 34*

FIG. 154.—*Average temperature and deformation, 4-inch limestone concrete protection, test No. 33*

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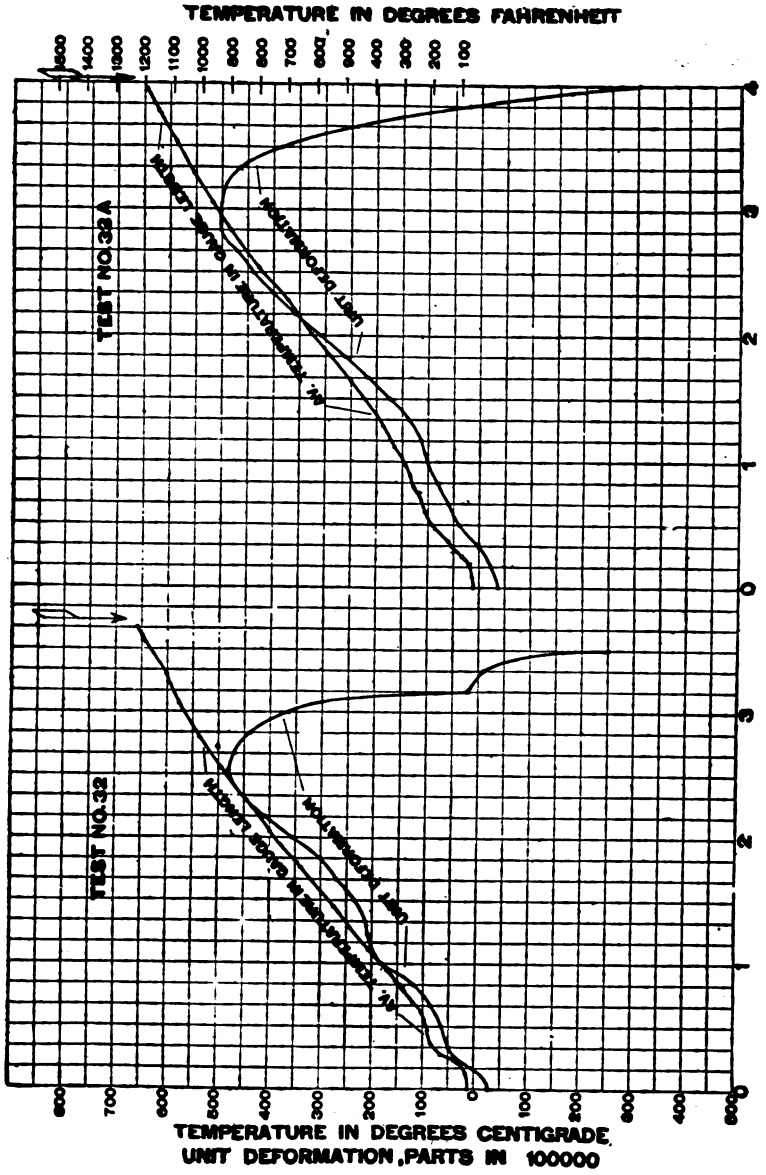

FIG. 153.—*Average temperature and deformation, 2-inch cinder concrete protections, test Nos. 32 and 32A*

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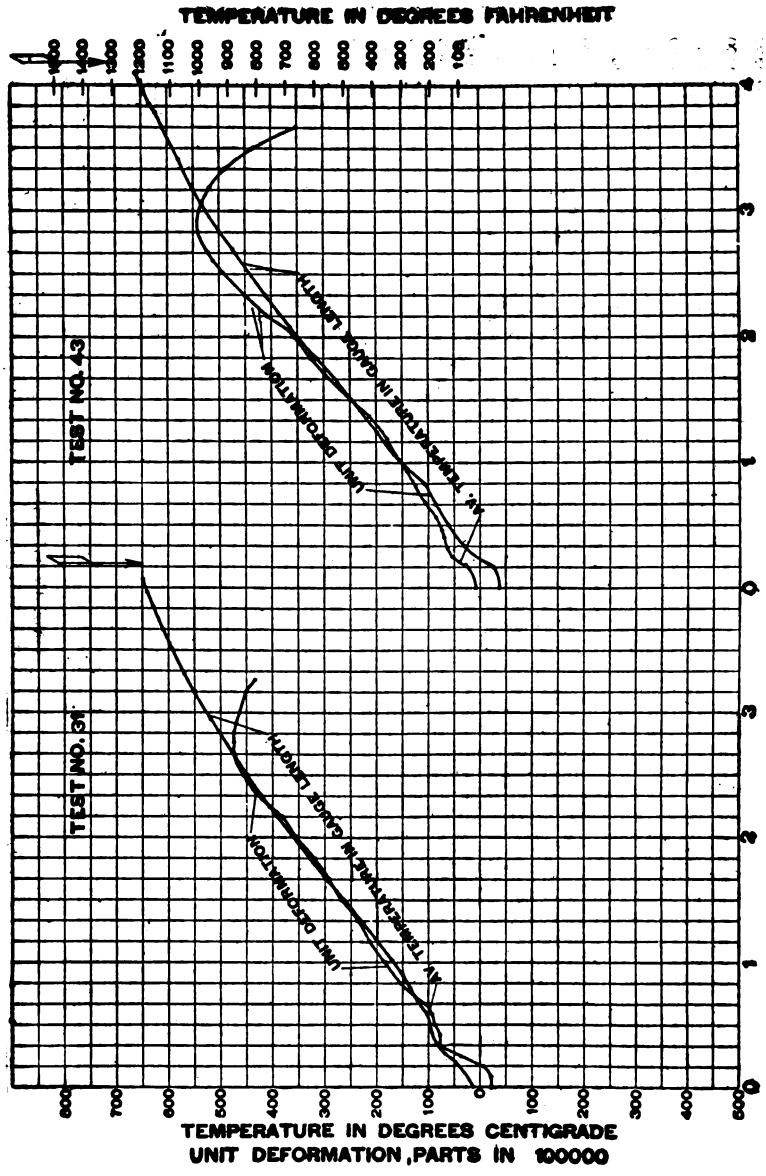

FIG. 152.—*Average temperature and deformation, 2-inch sandstone concrete protections, test Nos. 31 and 43*

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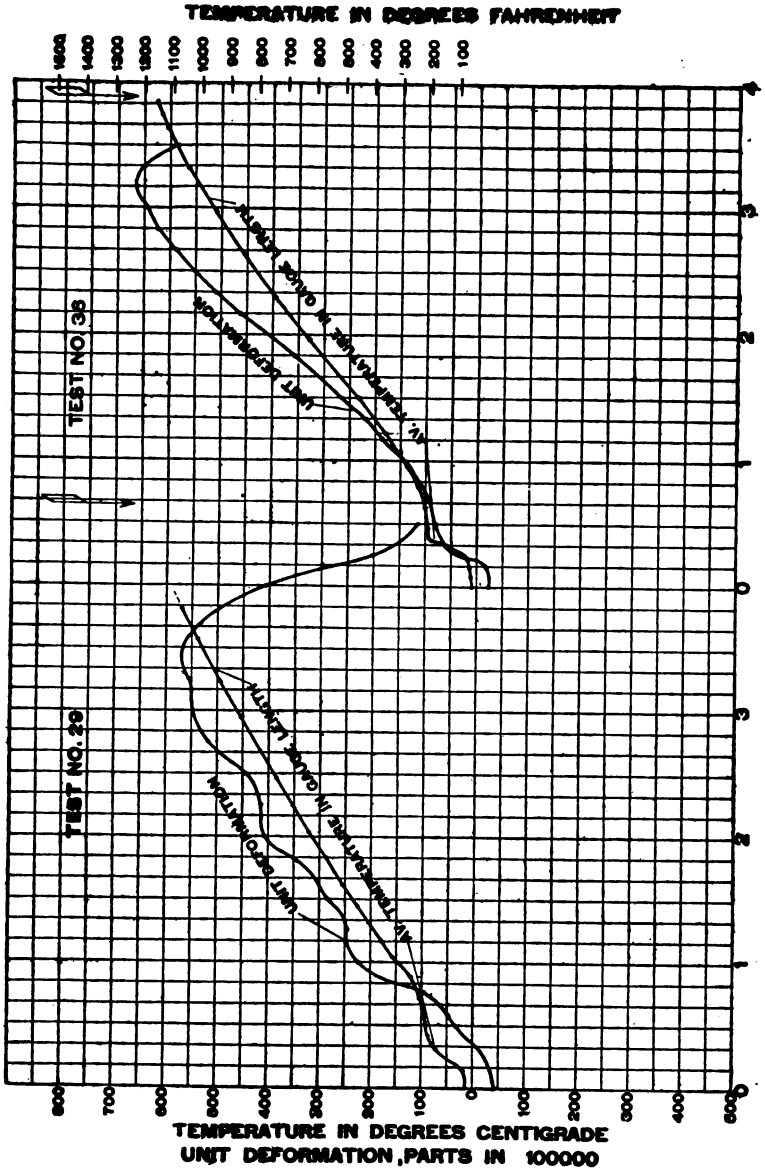

FIG. 151.—*Average temperature and deformation, 2-inch trap-rock concrete protections, test Nos. 29 and 36*

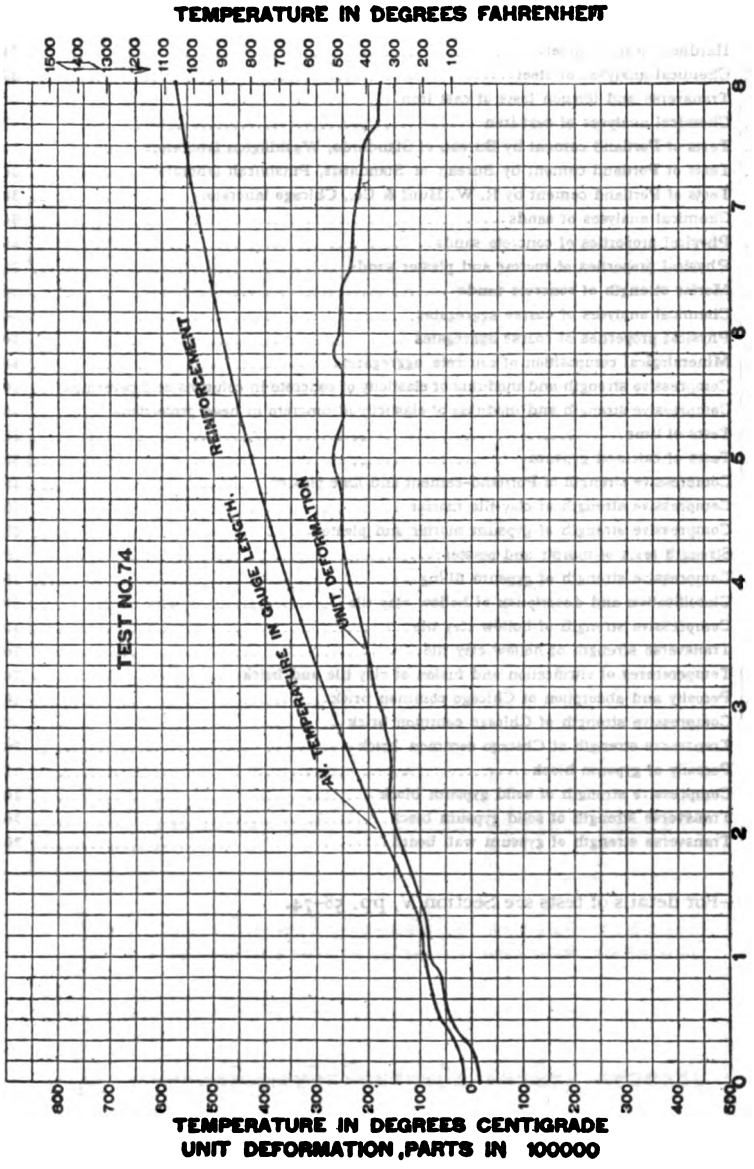

FIG. 171.—*Average temperature and deformation, round, hooped, limestone concrete column, test No. 74*

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APPENDIX D.—Tables of Auxiliary Tests of Materials

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38	Compressive strength of solid gypsum block	363
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40	Transverse strength of gypsum wall board	364

NOTE.—For details of tests see Section V, pp. 58–74.

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TABLE 5.—TENSION TESTS OF STEEL

Column Test No.	Specimen No.	SECTION	Location of Specimen	Yield Point, Lb. per Sq. In.	Ultimate Strength, Lb. per sq. In.	Elongation in 2 In., Percent	Reduction in Area, Percent
1	T-1	Rolled H	Edge of flange	40025	62600	41.0	57.8
1	T-1A	do	Junction of web and flange	37500	60000	37.0	61.5
1	T-1B	do	Center of web	42300	62600	40.0	52.8
14	T-14	do	do	.....	76300	32.0	61.0
28	T-28	do	do	34850	57400	37.5	65.6
28A	T-28A	do	do	37500	68250	32.5	59.2
30	T-30	do	do	34100	57600	25.0	67.0
31	T-31	do	do	38000	58600	29.0	60.0
32	T-32	do	do	42900	69400	17.5	61.0
33	T-33	do	do	35000	59000	27.5	66.0
33A	T-33A	do	do	37200	71500	24.0	54.0
34	T-34	do	do	48050	69800	25.0	55.0
34A	T-34A	do	do	32200	62700	21.0	66.0
35	T-35	do	do	34000	66000	16.0	63.0
48	T-48	do	do	39200	60800	25.0	61.0
49	T-49	do	Edge of flange	42900	64700	40.0	60.3
49	T-49A	do	Junction of web and flange	37750	60400	40.0	68.5
49	T-49B	do	Center of web	44700	62300	35.0	54.2
67A	T-67A	do	Edge of flange	41260	62230	43.0	62.8
68	T-68	do	do	35300	56720	47.0	64.3
16	T-16	Plate and Angle	Edge of angle	42850	59800	27.0	62.0
23	T-23	do	Center of plate	35500	58900	28.5	56.0
26	T-26	do	do	32100	59600	26.5	65.0
50A	T-50A	do	do	37650	60200	28.0	53.0
51	T-51	do	Edge of angle	34200	60500	25.0	61.0
105	T-105	do	Center of plate	29450	55300	27.5	69.0
24	T-24	Plate and Channel	Center of channel web	.....	61300	31.0	58.0
25	T-25	do	do	40500	67600	23.0	54.0
29	T-29	do	do	36600	58900	32.0	66.0
53	T-53	do	do	30000	62500	26.0	57.0
65	T-65	do	Edge of channel flange	37850	57650	39.0	58.5
65	T-65-1	do	Junction of flange and web	.....	51600	34.0	43.1
107	T-107	do	Center of channel web	46300	63400	27.0	54.0

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TABLE 5.—TENSION TESTS OF STEEL—Continued

Column Test No.	Specimen No.	SECTION	Location of Specimen	Yield Point, Lb. per Sq. In.	Ultimate Strength, Lb. per Sq. In.	Elongation in 2 In., Percent	Reduction in Area, Percent
18	T-18	Latticed Channel	Center channel web	44400	70900	23.0	55.0
40	T-40	do	do	40500	62200	28.0	63.0
19	T-19	Z-Bar and Plate	Center of web	35700	60500	25.0	55.0
25	T-25	do	do	32900	52200	28.0	65.0
32A	T-32A	do	do	35050	57200	30.5	79.0
41	T-41	do	do	31000	54400	23.0	62.0
42	T-42	do	do	44300	69700	17.5	58.0
55	T-55	do	do	43000	65000	19.0	55.0
56	T-56	do	do	37100	54200	25.0	63.0
20	T-20	I-Beam and Channel	Channel web	44800	64800	26.0	55.0
31	T-31	do	do	40500	62200	28.0	52.0
57	T-57	do	do	42100	65300	24.0	56.0
59	T-59	do	do	44000	63400	19.0	55.0
22	T-22	Latticed Angle	Edge of angle	42900	63700	18.0	63.0
46	T-46	do	do	47500	66300	35.0	64.0
8	T-8	Star	Edge of angle	39600	60300	24.0	62.0
45	T-45	do	do	43200	61100	26.5	64.0
70	70	Vertically Reinforced Concrete.	1 in. sq. deformed reinforcing bar	*34200	59500	22.8	63.4
70	70A	do	do	27340	47850	30.3	66.7
72	72	do	do	*36000	54000	26.8 (a)	62.0
72	72A	do	do	27620	46500	31.8	67.3
73	73-1	do	do	*35300	56000	27.0	62.5
73	73-1A	do	do	*35000	53750	25.8	64.5
73	73-2	do	do	*27700	46650	33.8	68.3
111	111-1	do	do	*35000	56000	24.2 (a)	64.5
111	111-1A	do	do	*35000	55500	26.5 (a)	64.5
111	111-2	do	do	35100	52570	22.5	67.8
112	112-1	do	do	*31000	48500	22.5	62.0
112	112-1A	do	do	*30000	48000	21.9	62.0
112	112-2	do	do	20670	34750	41.0	77.2

(a) Elongation was measured in 8-in. gage length instead of 2 in.
*Tested on reduced machined sections with shouldered ends.

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TABLE 5.—TENSION TESTS OF STEEL—Concluded

Column Test No.	Specimen No.	SECTION	Location of Specimen	Yield Point, Lb. per Sq. In.	Ultimate Strength, Lb. per Sq. In.	Elongation in 2 In., Percent	Reduction in Area, Percent
74	74-1	Hooped Reinforced Concrete	¾ in. sq. deformed reinforcing bar.	*40000	57500	28.3	64.2
74	74-1A	do.	do.	*36320	56700	31.0	64.6
74	74-2	do.	do.	36870	51690	35.8	67.7
74	74-3	do.	do.	*38800	54700	29.3	68.9
74	74-3A	do.	do.	40000	53500	31.0	64.7
74	74-4	do.	do.	*40850	59350	21.8	60.5
74	74-4A	do.	do.	*42800	59200	30.3	66.0
75	75-1	do.	do.	*37800	57700	29.5	64.4
75	75-1A	do.	do.	*39800	57500	29.8	65.8
75	75-2	do.	do.	37750	52870	35.8	64.6
113	113-1	do.	do.	*39800	58700	29.3	63.2
113	113-1A	do.	do.	*39800	59200	30.3	66.2
113	113-2	do.	do.	37950	54950	31.3	64.8
...	Q-1	do.	¼ in. high carbon wire hooping...	55500	86900	....	....
...	Q-2	do.	do.	49000	87600	....	....
...	Q-3	do.	do.	54000	87600	....	....
...	K-1	Ties in partly protected columns.	½ in. plain round bar.....	36680	58700	29.0	68.4
...	K-2	do.	do.	33930	54600	33.0	66.2
...	K-3	do.	do.	33000	48200	30.3	72.2
...	L-1	do.	⅜ in. plain round bar.....	40420	57700	27.5	68.9
...	L-2	do.	do.	40230	56950	27.3	70.5
...	L-3	do.	do.	34400	56170	28.5	70.7
...	M-1	Ties in vertically reinforced concrete columns.....	¼ in. plain round bar.....	41150	58500	....	68.5
...	M-2	do.	do.	41410	60000	....	67.4
...	M-3	do.	do.	41000	62500	....	67.8
...	M-4	do.	do.	44000	59100	35.0	70.4
...	M-5	do.	do.	43600	61200	36.0	68.6
...	M-6	do.	do.	43050	57500	38.0	71.6
...	N-1	Ties in concrete protections.	No. 5 (B. & S. ga.) bright basic wire	.....	76200	....	67.5
...	N-2	do.	do.	.....	73800	....	67.5
...	N-3	do.	do.	.....	75500	....	67.5

* Tested on reduced machined sections with shouldered ends.

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TABLE 6.—TENSION TESTS OF METAL LATH

Each value is the average of three tests

Specimen No.	Kind of Lath	Width of Specimen	Ultimate Strength, Lb. per In. Width
R-1	No. 24 Gage Expanded Metal	3 in.	150
R-2	No. 24 High-Ribbed Metal Lath	2½ in.	520
R-3	Woven Wire Lath	3 in. (7 strands)	256

TABLE 7.—HARDNESS TESTS OF STEEL

Column Test No.	Specimen No.	Section	Location of Specimen	Brinnell Hardness No.
2	H-2	Plate and Angle	Edge of angle	113
17	E-17	do	do	121
37	E-37	do	do	118
51A	E-51A	do	do	118
103	H-103	do	do	121
104	H-104	do	do	125
105	H-105	do	do	121
110	H-110	do	do	126
60	H-60	Latticed angle	do	123
				Scleroscope Hardness No.
3	HC-3	Plate and Channel	Center of channel web	23
3	HP-3	do	Plate	24
38	H-38	do	do	21
107	H-107	do	Center of channel web	24
66	H-66	Latticed channel	Center of web	23
5	H-5	Z-bar and Plate	Plate	26
41	H-41	do	do	22
56	H-56	do	do	24
6	H-6	I-beam and Channel	Channel web	26
42	H-42	do	Center of I-beam web	22
57	H-57	do	do	22
58	H-58	do	do	22
7	H-7	Latticed angle	Edge of angle	25
26	H-26	do	do	20
60	H-60	do	do	20
61	R-61	do	do	22

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TABLE 8.—CHEMICAL ANALYSES OF STEEL

Column Test No.	Specimen No.	Carbon	Manganese	Phosphorus	Sulphur	Silicon
1	1	0.20	0.54	0.010	0.021	0.03
1	1A	0.18	0.54	0.015	0.028	0.01
1	1B	0.21	0.54	0.012	0.025	0.04
14	14	0.16	0.72	0.023	0.034	0.03
65	65	0.18	0.40	0.011	0.036	0.04
65	65-1	0.20	0.41	0.010	0.033	0.05
65	65-2	0.21	0.42	0.006	0.029	0.04
68	68	0.12	0.57	0.010	0.022	0.02
22	Rivets	0.09	....	0.007	0.050	....
46	Rivets	0.11	....	0.024	0.064	....

TABLE 9.—TRANSVERSE AND TENSION TESTS OF CAST IRON

Column Test No.	Specimen No.	Modulus of Rupture, Lb. per Sq. In.	*Ultimate Tensile Strength, Lb. per Sq. In.
9	T-9	47800	23330
10	T-10	45800	29470
11	T-11	43500	23900
27	T-27	64700	26700
47	T-47	.....	23050
62	T-62	56100	28800
**16A	1CIA	.....	17600
114 and	1CIB	39400	18300
115	1CIC	36200	17100
do	2CIA	45500	20300
do	2CIB	46300	21700
do	2CIC	45700	21600

*Tension specimens cut from ends of transverse specimens after test.
**Specimens cut from duplicate test column.

TABLE 10.—CHEMICAL ANALYSIS OF CAST IRON

Column Test No.	Specimen No.	Carbon	Manganese	Phosphorus	Sulphur	Silicon	Graphite
9	T-9	0.61	0.47	0.57	0.090	1.86	2.66
10	T-10	0.67	0.50	0.57	0.090	1.86	2.77
11	T-11	....	....	....	0.087	....	....
27	T-27	....	....	....	0.086	....	....
62	T-62	....	....	....	0.082	....	....

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TABLE 11.—TESTS OF PORTLAND CEMENT

By Bureau of Standards' Washington Laboratory

DATE OF REPORT	September 22, 1916							Dec. 19, 1916
SAMPLE No.	1	2	3	4	5	B–1	B–2	12*
Chemical Analysis:								
Silica (SiO_2)	20.34%	19.46%	20.36%	20.28%	20.40%	20.46%	20.82%	20.82%
Oxide of Iron (Fe_2O_3)	2.72%	2.70%	2.60%	2.70%	2.80%	2.75%	2.52%	2.10%
Oxide of Aluminum (Al_2O_3)	6.99%	7.18%	7.24%	6.86%	6.74%	6.15%	6.39%	7.14%
Lime (CaO)	63.68%	62.12%	63.42%	64.32%	63.24%	62.80%	64.68%	59.50%
Magnesia (MgO)	1.13%	1.21%	0.90%	1.18%	1.12%	1.22%	1.28%	3.46%
Sulphuric Anhydride (SO_3)	1.34%	1.44%	1.45%	1.45%	1.46%	1.38%	1.45%	1.50%
Loss of Ignition	4.53%	3.43%	3.17%	3.17%	3.22%	3.40%	2.20%	4.48%
Insoluble Residue	0.10%	0.13%	0.17%	0.20%	0.13%	0.16%	0.13%	0.70%
Specific Gravity:								
As Received	3.06	3.09	3.08	3.08	3.08	3.10	3.12	.....
After Ignition	3.17	3.18	3.16	3.22	3.15	.....	.....	3.17
Fineness:								
Passing 110-Mesh Sieve	94.8%	94.4%	97.0%	96.0%	97.0%	97.4%	96.4%	99.2%
Passing 200-Mesh Sieve	77.6%	76.0%	77.4%	78.4%	77.0%	78.5%	78.1%	75.0%
Tensile Strength, Lb. per Sq. In.:								
Neat, 7 Days	434	552	551	587	680	622	736	435
	471	512	641	645	527	564	575	510
	575	619	562	604	577	670	587	425
Neat, 28 Days	730	690	670	690	545	715	725	740
	635	655	710	640	670	800	675	625
	585	595	730	575	705	700	665	675
1 Cement: 3 Sand—								
7 Days	178	227	222	182	228	121	193	180
	178	229	182	207	229	177	247	140
	169	206	206	213	243	209	238	165
28 Days	280	295	325	325	320	265	365	375
	310	300	310	335	340	280	380	380
	260	265	310	330	290	325	330	340
Time of Set, hr.—min.:								
Initial	7–30	7–15	7–30	7–10	7–20	7–30	7–25	7–0
Final	11–00	11–30	11–55	11–45	11–40	11–50	11–00	10–30
Soundness: Pats—								
28 Days in Air	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O K.
28 Days in Water	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.
5 Hr. in Steam	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.
Water Used:								
Neat	25.0%	25.0%	25.0%	25.0%	25.0%	25.0%	25.0%	23.0%
1:3 Mortar	10.7%	10.7%	10.7%	10.7%	10.7%	10.7%	10.7%	10.3%

*Portland cement used in pipe column, Test No. 12.

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TABLE 12.—TESTS OF PORTLAND CEMENT

By Bureau of Standards' Pittsburg Laboratory

DATE OF REPORT	May 1, 1917			June 19, 1917		Nov. 19, 1918	
SAMPLE No.	H-1	H-2	H-3	H-5	H-6	H-7	H-8
Specific Gravity:							
As Received	3.15	3.18	3.18	3.10	3.11	3.11	3.12
After Ignition	3.16	3.18	3.19	.....	.....	.....	.....
Fineness:							
Passing 100-Mesh Sieve	95.0%	96.0%	95.4%	94.0%	94.2%	.....	.....
Passing 200-Mesh Sieve	77.2%	77.7%	77.0%	76.2%	76.4%	78.8%	78.6%
Tensile Strength, Lb. per Sq. In.:							
Neat, 7 Days	516	598	695	621	450	.....	.....
	505	550	685	474	521	.....	.....
	521	510	650	530	543	.....	.....
Neat, 28 Days	725	750	700	585	661	.....	.....
	735	688	690	600	575	.....	.....
	650	681	776	570	691	.....	.....
1 Cement: 3 Sand—							
7 Days	190	204	261	203	205	300	310
	208	242	260	225	225	260	300
	245	223	269	212	239	300	305
28 Days	358	282	322	315	349	410	440
	371	332	326	331	381	430	470
	376	345	340	382	352	400	460
Time of Set, hr.—min.:							
Initial	7–0	7–0	7–0	Over 1 hr.	Over 1 hr.	3–50	3–50
Final	Over 10 hrs.	Over 10 hrs.	Over 10 hrs.	Over 10 hrs.	Over 10 hrs.	8–50	9–0
Soundness: Pats—							
5 Hr. in Steam	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.
Water Used:							
Neat	24.0%	24.0%	24.0%	24.0%	24.0%	.....	.....
1:3 Mortar	10.5%	10.5%	10.5%	10.5%	10.5%	.....	.....

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TABLE 13.—TESTS OF PORTLAND CEMENT

By R. W. Hunt & Company

DATE OF REPORT	May 2, 1917		
SAMPLE No.	H-1	H-2	H-3
Specific Gravity:			
As Received	3.14	3.13	3.13
Fineness:			
Passing 100-Mesh Sieve	95.4%	96.0%	96.1%
Passing 200-Mesh Sieve	76.4%	78.0%	77.2%
Tensile Strength, Lb. per Sq. In.:			
Neat, 24 Hr.	305	355	345
	320	330	330
	335	350	300
Neat, 7 days	690	625	675
	640	640	650
	700	700	695
Neat, 28 Days	700	700	695
	730	765	710
	765	725	725
1 Cement, 3 Sand:			
7 Days	310	330	300
	330	310	320
	335	345	325
28 Days	410	385	410
	380	395	400
	405	410	385
Time of Set, hr.—min.:			
Initial	5-45	5-40	5-40
Final	8-05	8-0	8-10
Soundness: Pats—			
5 Hr. in Steam	O. K.	O. K.	O. K.
Water Used:			
Neat	23.0%	23.0%	23.0%
1 : 3 Mortar	10.3%	10.3%	10.3%

TABLE 14.—CHEMICAL ANALYSES OF SANDS

SAND	Insoluble in Acid	Al_2O_3 and Fe_2O_3	CaO	MgO	CO_2	*Loss on Ignition
Fox River	54.51	1.34	14.90	8.24	21.22	.....
Joliet	52.10	1.19	15.46	8.97	23.06	.....
Meramec River	98.66	0.58	0.14	0.06	0.11	.....
Long Island	97.98	0.96	0.04	0.18	0.25	.....
Pelee Island	64.70	1.66	17.03	1.34	15.68	.....
Plum Island	98.18	0.66	0.16	0.12	0.18	.....
Cambridge	96.93	1.63	0.17	0.90	0.25	0.97
Coarse lake	83.53	6.69	3.31	1.21	3.30	3.63
Fine lake	87.69	5.38	2.17	0.93	1.95	2.28
Beach	85.45	5.60	2.35	1.02	2.29	2.79
Bank	91.30	5.18	0.09	0.51	0.22	0.54

*Not including CO_2.

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TABLE 15.—PHYSICAL PROPERTIES OF CONCRETE SANDS

SAND.	Specific Gravity	Weight per Cu. Ft. (Dry), Lb.	Computed Voids, Per cent	Granular Analysis—Percent Passing Sieve No.											
				3	4	6	8	10	14	20	28	35	48	65	100
Fox River	2.62	110.0	33.0	99.4	93.5	86.9	79.6	71.4	63.7	53.2	37.1	22.9	11.0	5.3	2.5
Joliet	2.62	96.3	41.0	98.4	95.0	89.5	82.7	73.6	64.5	52.5	33.6	15.0	3.4	0.8	0.3
Meramec River	2.63	100.0	39.0	99.4	98.8	96.4	92.5	87.6	83.0	78.4	71.7	56.2	22.1	5.5	0.9
Long Island	2.66	97.5	41.0	99.5	98.2	95.5	89.0	80.3	71.8	61.5	47.6	32.5	15.2	6.6	2.4
Pelee Island	2.66	112.0	33.0	100.0	100.0	93.8	82.9	69.3	58.7	48.6	37.8	27.9	15.6	8.6	3.1
Plum Island	2.63	92.1	44.0	100.0	99.8	99.5	98.6	96.8	89.0	69.2	34.2	7.2	0.8	0.3	0.1
Cambridge	2.65	109.4	34.0	91.5	88.6	85.2	81.0	75.8	70.0	60.5	46.3	32.9	16.6	9.0	5.9
Sieve Opening, In	.....	.....	.....	0.263	0.185	0.131	0.093	0.065	0.046	0.0328	0.0232	0.0164	0.0116	0.0082	0.0058

TABLE 16.—PHYSICAL PROPERTIES OF MORTAR AND PLASTER SANDS

SAND	Specific Gravity	Weight per cu. ft. (Dry), Lb.	Computed Voids, Percent	Granular Analysis—Percent Passing Sieve No.							
				6	10	20	40	60	80	100	200
Coarse Lake	2.62	104.6	36.0	98.5	94.0	72.9	42.5	12.6	5.4	1.9	0.7
Fine Lake	2.62	100.3	38.0	99.5	98.3	94.2	76.1	42.6	16.4	10.4	0.6
Beach	2.63	91.7	44.0	99.0	97.0	90.7	75.5	48.8	31.3	17.5	0.3
Bank	2.64	99.6	38.0	99.9	99.9	99.7	98.5	89.7	47.5	20.9	.....
Sieve Opening, In	.....	.....	.....	0.131	0.065	0.0328	0.015	0.0087	0.0065	0.0058	0.0029

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TABLE 17.—MORTAR STRENGTH TESTS OF CONCRETE SANDS

Proportion: 1 part Tidewater Portland cement, 3 parts sand (by weight)

Test specimens stored in water. Each result is the average of three tests.

SAND	Percent Water	Age, Days	Tensile Strength, Lb. per Sq. In.	Compressive Strength, Lb. per Sq. In.
Fox River	10.0	7	283	3390
Fox River	10.0	28	437	4980
Joliet	12.0	7	397	1347
Joliet	12.0	28	379	2920
Meramec River	13.5	7	171	921
Meramec River	13.5	28	346	1540
Long Island	13.0	7	176	954
Long Island	13.0	28	303	1910
Pelee Island	11.0	7	245	763
Pelee Island	11.0	28	412	3320
Plum Island	12.0	7	258	1609
Plum Island	12.0	28	269	1621

TABLE 18.—CHEMICAL ANALYSES OF COARSE AGGREGATES

AGGREGATE	Insoluble in Acid	Silica, Soluble	Al_2O_3 and R_2O_3	CaO	MgO	CO_2	SO_3	Loss* on Ignition
Chicago limestone	5.00	.....	0.44	29.16	20.40	45.35	.....	1.00
Joliet gravel	21.28	0.24	0.82	22.30	16.29	35.70	.....	1.16
Meramec R. gravel	96.06	0.50	2.36	Trace	Trace	.....	None	0.92
New York trap rock	86.08	0.24	8.22	1.05	1.93	.....	.....	.....
Rockport granite	97.31a	.....	.....	.....	.....	.....	.....	.....
Cleveland sandstone	96.59b	.....	.....	.....	.....	.....	.....	.....
Hard coal cinders	90.88	0.13	0.62	0.41	0.25	.....	0.03	6.97
Westfield trap rock	84.63	1.48	7.48	1.63	2.85	2.65	.....	0.19

* Other than CO_2.
(a) Soluble content too small to warrant further analysis. Tests indicate a normal granite.
(b) Normal sandstone.

TABLE 19.—PHYSICAL PROPERTIES OF COARSE AGGREGATES

AGGREGATE	Apparent Specific Gravity	Weight per Cu. Ft. (Dry), Lb.	Computed Voids, Percent	Percent Passing Sieves Size of Openings in In.						
				1½	1	¾	½	¼	0.131	0.093
Chicago limestone	2.61	86.50	46.9	100.0	98.5	76.5	33.0	6.0	0.14	0.11
Joliet gravel	2.61	101.25	37.9	100.0	95.7	83.5	56.5	6.5	.....	.....
Meramec R. gravel	2.45	95.0	38.0	100.0	98.0	91.5	62.0	19.0	6.60	3.60
New York trap rock	2.87	95.6	46.6	100.0	98.3	83.3	50.8	11.8	0.20	0.10
Rockport granite	2.61	87.25	46.3	100.0	100.0	100.0	86.5	29.0	7.30	1.60
Cleveland sandstone	2.15	78.50	41.6	98.7	83.7	45.0	20.0	9.0	7.30	7.00
Hard coal cinders	1.52	50.75	46.5	98.8	93.5	89.0	68.7	37.3	36.0	34.0
Westfield trap rock	2.94	90.0	51.0	100.0	100.0	100.0	100.0	58.3	38.8	19.4

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TABLE 20.—MINERALOGICAL COMPOSITION OF CONCRETE AGGREGATES

AGGREGATE	Calcite, $CaCO_3$	Dolomite, $CaMg(CO_3)_2$	Clay, Hydrous Aluminum Silicates	Limonite (Iron Rust), $2Fe_2O_3.3H_2O$	Chert, $SiO_2.XH_2O$	Quarts, SiO_2	Feldspars, $KAlSi_3O_8$ $NaAlSi_3O_8$ $CaAl_2Si_2O_8$	Ferromagnesians, (Fe, Mg, Ca) Al_2SiO_6 (Fe, Mg, Ca) SiO_3	Magnetite, Fe_3O_4	Other Minerals. Amounts, Where not Given, Are Less Than 1 Percent
Fox River sand	6	38	(2 in calcite and dolomite)	0.5	7	32	11	4	0.5	white mica, garnet titanite
Joliet sand	5	42	(3 in calcite and dolomite)	trace		42	7	3	trace	garnet, cinders
Meramec River sand	1			trace	16	83				
Long Island sand	trace		1 (slate)			82	14	2	0.7	garnet, cinders
Pelee Island sand	27	5	4		4	47	9	3	0.5	garnet, muscovite corundum
Plum Island sand	trace					83	14	2	0.3	garnet, muscovite
Chicago limestone		94.5	5	0.5						
Joliet gravel		86	(5 in dolomite)			7	5	2		
Meramec River gravel	trace			trace	98	2				
New York trap rock						trace	71	26	2	apatite
Rockport granite			trace	trace		35	59	5	trace	zircon
Cleveland sandstone			1.5	0.5		98				
Hard coal cinders										85 slag and cinder 10 anthracite 5 ash

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TABLE 21.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN COLUMNS AND COVERINGS

Cylinder No.	Kind of Concrete	Age, Days	Percent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
22-1	1:2:4 Chicago limestone	28	.....	912	.....	.....	.....
22-2	do	29	.....	1049	.....	.....	.....
22-3	do	407	.....	1888	2,060,000	2,490,000	2,780,000
22-4	do	407	.....	1780	2,390,000	2,400,000	2,310,000
28-1	do	28	.....	2935	.....	.....	.....
28-3	do	28	.....	3308	.....	.....	.....
28-2	do	438	.....	3890	.....	.....	5,270,000
28-4	do	438	.....	3470	4,470,000	4,620,000	4,400,000
28A-1	do	28	.....	1738	.....	.....	.....
28A-3	do	28	.....	1647	.....	.....	.....
28A-2	do	439	.....	2370	4,300,000	3,960,000	3,550,000
28A-4	do	439	.....	2213	.....	3,960,000	3,500,000
33-1	do	25	12.1	1489	.....	.....	.....
33-3	do	25	12.1	1728	.....	.....	.....
33-2	do	452	12.1	1956	2,660,000	2,300,000	1,980,000
33-4	do	452	12.1	2273	4,000,000	3,720,000	3,350,000
33A-1	do	29	12.9	1255	.....	.....	.....
33A-3	do	29	12.9	1676	.....	.....	.....
33A-2	do	456	12.9	1574	2,980,000	2,680,000	2,450,000
33A-4	do	456	12.9	1881	3,350,000	3,100,000	2,980,000
70-1	do	28	.....	2027	.....	.....	.....
70-3	do	28	.....	1778	.....	.....	.....
70-2	do	434	.....	3182	4,280,000	3,950,000	3,880,000
70-4	do	434	.....	2743	3,750,000	3,730,000	3,490,000
72-1	do	29	13.1	1010	.....	.....	.....
72-3	do	29	12.7	1582	.....	.....	.....
72-2	do	521	13.1	1656	2,730,000	2,800,000	2,250,000
72-4	do	521	12.7	2114	.....	.....	2,970,000
74-1	do	28	12.0	704	.....	.....	.....
74-3	do	28	11.8	739	.....	.....	.....
74-2	do	523	12.0	922	1,420,000	760,000	280,000
74-4	do	523	11.8	935	1,760,000	1,160,000	490,000
101-5	do	28	12.7	1495	.....	.....	.....
101-6	do	520	12.7	2485	7,970,000	5,990,000	5,080,000
102-1	do	28	12.2	1455	.....	.....	.....
102-2	do	514	12.2	2363	4,050,000	3,760,000	3,740,000
111-1	do	28	12.5	1554	.....	.....	.....
111-2	do	520	12.5	2735	5,000,000	4,320,000	4,450,000
112-5	do	28	10.8	1493	.....	.....	.....
112-6	do	521	10.6	2042	3,640,000	3,400,000	3,350,000
19-1	1:3:5 Chicago limestone	29	.....	2056	.....	.....	.....
19-2	do	29	.....	2168	.....	.....	.....
19-3	do	414	.....	2561	4,000,000	3,940,000	3,790,000
19-4	do	414	.....	1880	3,350,000	3,160,000	3,120,000
35-1	do	32	12.8	1284	.....	.....	.....
35-3	do	32	12.8	1618	.....	.....	.....
35-2	do	505	12.8	1540	3,270,000	2,940,000	2,520,000
35-4	do	505	12.8	1466	4,070,000	3,760,000	3,470,000
41-1	do	29	13.0	1355	.....	.....	.....
41-3	do	29	13.4	1132	.....	.....	.....
41-2	do	452	13.0	1485	2,830,000	2,520,000	2,300,000
41-4	do	452	13.4	1313	2,590,000	2,450,000	2,200,000
42-1	do	28	13.3	1169	.....	.....	.....
42-3	do	27	12.6	1042	.....	.....	.....
42-2	do	450	13.3	1943	3,450,000	3,180,000	3,180,000
42-4	do	450	12.6	1216	2,350,000	2,000,000	1,640,000
55-1	do	29	13.7	718	.....	.....	.....
55-3	do	29	15.2	577	.....	.....	.....
55-2	do	491	13.7	794	1,330,000	490,000	.....
55-4	do	483	15.2	678	1,630,000	.....	.....
56-1	do	28	13.3	811	.....	.....	.....
56-3	do	28	12.9	614	.....	.....	.....
56-2	do	492	13.3	962	1,280,000	400,000	.....
56-4	do	492	12.9	809	1,280,000	760,000	.....
57-1	do	28	12.7	912	.....	.....	.....
57-3	do	28	12.9	947	.....	.....	.....
57-2	do	488	12.7	921	1,680,000	900,000	350,000

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TABLE 21.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN COLUMNS AND COVERINGS—Continued

Cylinder No.	Kind of Concrete	Age, Days	Per-cent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
57-4	1:3:5 Chicago limestone	488	12.9	1040	2,080,000	1,550,000	1,000,000
76-1	do	59	13.7	528	190,000		
76-2	do	59	13.7	688	340,000	240,000	
77-1	do	59	13.7	842	1,360,000	960,000	
77-2	do	59	13.7	506	210,000		
105-1	do	28	13.9	767			
105-4	do	29	13.9	813			
106-2	do	532	13.9	820	1,770,000	1,200,000	
106-3	do	532	13.9	946	2,230,000	1,460,000	1,000,000
16-1	1:2:4 New York trap	29		1023			
16-2	do	29		1163			
16-3	do	416		2512	4,750,000	4,340,000	3,880,000
16-4	do	416		1219	1,100,000	650,000	330,000
18-1	do	28		1158			
18-2	do	28		1764			
18-3	do	419		1529	1,900,000	1,570,000	1,080,000
18-4	do	419		1884	3,320,000	2,600,000	1,840,000
29-1	do	29	13.1	1494			
29-3	do	29	13.6	1239			
29-2	do	437	13.1	1807	2,570,000	2,390,000	2,210,000
29-4	do	437	13.6	1985	3,140,000	2,870,000	2,640,000
36-1	do	29	12.2	1490			
36-3	do	29	12.0	1548			
36-2	do	445	12.2	2284	2,750,000	2,740,000	2,000,000
36-4	do	445	12.0	2118	3,100,000	2,980,000	2,650,000
37-1	do	30	12.6	995			
37-3	do	30	12.7	1668			
37-2	do	504	12.6	2278	2,780,000	2,620,000	2,450,000
37-4	do	504	12.7	1108	820,000	530,000	350,000
40-1	do	28	11.2	1797			
40-3	do	28	11.1	1780			
40-2	do	502	11.2	2180	3,140,000	2,960,000	2,750,000
40-4	do	502	11.1	2300	4,170,000	3,480,000	3,150,000
46-1	do	28	13.9	1418			
46-3	do	29	13.9	1641			
46-2	do	455	13.9	1659	1,870,000	1,700,000	1,460,000
46-4	do	455	13.9	2062	2,730,000	2,510,000	2,300,000
60-1	do	27	14.6	948			
60-3	do	27	14.6	1028			
60-2	do	490	14.6	1490	2,510,000	2,140,000	1,810,000
60-4	do	490	13.8	1039	1,940,000	1,530,000	1,200,000
71-1	do	29	13.7	1572			
71-3	do	29	13.0	957			
71-2	do	451	13.7	1446	2,450,000	1,970,000	1,650,000
71-4	do	451	13.0	1109	1,000,000	400,000	230,000
73-1	do	28		2239			
73-3	do	28		2484			
73-2	do	443		2658	4,180,000	3,980,000	3,720,000
73-4	do	443		3224	3,000,000	4,060,000	3,830,000
75-1	do	28	10.0	1361			
75-3	do	28	10.3	1756			
75-2	do	462	10.0	1918	1,500,000	1,610,000	1,740,000
75-4	do	462	10.3	2318	2,790,000	2,770,000	2,700,000
101-3	do	28	13.2	1307			
101-4	do	541	13.2	2115	4,850,000	4,480,000	4,090,000
102-5	do	28	11.9	1510			
102-6	do	516	11.9	2692	3,020,000	3,060,000	3,070,000
104-1	do	32	11.1	978			
104-2	do	527	11.1	1542	3,900,000	3,510,000	2,350,000
20-1	1:3:5 New York trap	30		1254			
20-2	do	30		1158			
20-3	do	415		1109	4,980,000	940,000	450,000
20-4	do	415		958	8,700,000	330,000	
21-1	do	28		702			
21-2	do	28		678			
21-3	do	416		874	590,000	270,000	
21-4	do	416		986	900,000	520,000	250,000
54-1	do	29	12.6	780			
54-2	do	488	12.6	1012	1,330,000	1,850,000	530,000

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TABLE 21.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN COLUMNS AND COVERINGS—Continued

Cylinder No.	Kind of Concrete	Age, Days	Per-cent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
15-1	1:2:4 Rockport granite	30	.....	1478	.....	.....	.....
15-2	do	30	.....	1181	.....	.....	.....
15-3	do	407	.....	1773	2,800,000	2,730,000	2,600,000
15-4	do	407	.....	2315	2,470,000	2,540,000	2,540,000
34-1	do	29	12.0	1638	.....	.....	.....
34-3	do	29	11.5	1306	.....	.....	.....
34-2	do	453	12.0	2211	2,800,000	2,570,000	2,390,000
34-4	do	453	11.5	1908	3,160,000	2,900,000	2,550,000
34A-1	do	28	12.7	853	.....	.....	.....
34A-3	do	28	10.7	1638	.....	.....	.....
34A-2	do	455	12.7	1157	2,060,000	1,750,000	1,350,000
34A-4	do	455	10.7	1838	2,680,000	2,670,000	2,566,000
103-1	do	28	13.0	998	.....	.....	.....
103-2	do	522	13.0	1645	3,620,000	3,100,000	2,980,000
113-1	do	32	12.2	1017	.....	.....	.....
113-2	do	527	12.2	1476	.....	2,480,000	2,220,000
50-1	1:2:5 Rockport granite	28	13.5	919	.....	.....	.....
50-3	do	28	13.5	621	.....	.....	.....
50-2	do	487	13.5	1062	2,150,000	1,800,000	1,350,000
50-4	do	487	13.5	1063	2,380,000	1,850,000	1,280,000
50A-1	do	28	11.5	734	.....	.....	.....
50A-3	do	28	11.5	660	.....	.....	.....
50A-2	do	509	11.5	1255	2,400,000	1,950,000	1,650,000
50A-4	do	509	11.5	1073	2,750,000	2,430,000	2,220,000
51-1	do	29	14.0	729	.....	.....	.....
51-4	do	29	14.0	708	.....	.....	.....
51-2	do	485	14.0	924	1,480,000	1,130,000	750,000
51-3	do	485	14.0	688	1,480,000	.....	.....
51A-1	do	29	14.1	683	.....	.....	.....
51A-4	do	29	14.1	822	.....	.....	.....
51A-2	do	507	14.1	877	2,170,000	1,260,000	640,000
51A-3	do	507	14.1	1272	2,270,000	2,230,000	2,000,000
31-1	1:2:4 Cleveland sandstone	30	14.0	2040	.....	.....	.....
31-3	do	30	14.2	1988	.....	.....	.....
31-2	do	501	14.0	2828	1,410,000	1,410,000	1,410,000
31-4	do	501	14.2	2531	1,230,000	1,230,000	1,200,000
43-1	do	29	14.0	746	.....	.....	.....
43-3	do	29	14.0	1855	.....	.....	.....
43-2	do	456	14.0	2128	1,420,000	1,290,000	1,230,000
43-4	do	456	14.0	2658	1,430,000	1,350,000	1,270,000
104-3	do	32	14.9	1535	.....	.....	.....
104-4	do	527	14.9	2720	2,270,000	2,020,000	1,670,000
44-1	1:3:5 Cleveland sandstone	29	14.7	1418	.....	.....	.....
44-3	do	29	14.7	661	.....	.....	.....
44-2	do	459	14.7	1662	4,130,000	3,040,000	2,050,000
44-4	do	459	14.7	1042	890,000	720,000	600,000
11-1	1:2:4 Joliet gravel	27	11.3	2365	.....	.....	.....
11-4	do	27	11.3	1764	.....	.....	.....
11-2	do	468	11.3	2190	3,030,000	3,100,000	3,120,000
11-3	do	468	11.3	2522	3,630,000	3,530,000	3,400,000
14-1	do	28	.....	1070	.....	.....	.....
14-2	do	28	.....	1192	.....	.....	.....
14-3	do	406	.....	2177	.....	3,670,000	3,470,000
14-4	do	406	.....	1268	3,380,000	2,940,000	2,620,000
30-1	do	28	.....	762	.....	.....	.....
30-3	do	28	.....	1908	.....	.....	.....
30-2	do	440	.....	898	2,250,000	1,550,000	620,000
30-4	do	440	.....	2690	3,240,000	3,210,000	3,200,000
38-1	do	29	10.8	1080	.....	.....	.....
38-3	do	29	10.8	1290	.....	.....	.....
38-2	do	451	10.8	1872	3,350,000	3,000,000	2,840,000
38-4	do	451	10.8	1721	3,500,000	3,220,000	3,130,000
101-1	do	28	11.0	1401	.....	.....	.....
101-2	do	520	11.0	2408	3,810,000	3,750,000	3,600,000
102-3	do	28	11.0	1720	.....	.....	.....
102-4	do	514	11.0	2951	4,890,000	4,370,000	4,130,000

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TABLE 21.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN COLUMNS AND COVERINGS—Concluded

Cylinder No.	Kind of Concrete	Age, Days	Percent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
112-1	1:2:4 Joliet gravel	28	9.7	2004			
112-2	do	521	9.7	2978	4,200,000	4,160,000	4,140,000
39-1	1:2:4 Meramec R. gravel	28	.....	2058			
39-3	do	28	.....	1220			
39-2	do	437	.....	2580	3,450,000	3,500,000	3,430,000
39-4	do	437	.....	1721	3,550,000	3,500,000	3,430,000
45-1	do	25	11.8	2056			
45-3	do	25	12.3	1987			
45-2	do	447	11.8	2707	4,600,000	4,370,000	4,040,000
45-4	do	447	12.3	2374	3,870,000	3,920,000	3,900,000
112-3	do	28	10.7	1719			
112-4	do	521	10.7	2952	3,310,000	3,510,000	3,600,000
17-1	1:1½:4½ Hard coal cinders	28	.....	671			
17-2	do	28	.....	1307			
17-3	do	408	.....	1691	1,720,000	1,500,000	1,330,000
17-4	do	408	.....	2072	1,900,000	1,770,000	1,670,000
32-1	1:2:5 Hard coal cinders	29	22.9	495			
32-3	do	29	22.9	728			
32-2	do	504	22.9	970	1,080,000	900,000	700,000
32-4	do	504	22.9	977	1,250,000	1,000,000	610,000
32A-1	do	32	17.3	628			
32A-3	do	32	18.5	759			
32A-2	do	498	17.3	950	900,000	720,000	500,000
32A-4	do	498	18.5	796	750,000	530,000	.....
47-1	do	28	22.1	949			
47-3	do	28	22.4	907			
47-2	do	447	22.1	1335	1,070,000	1,050,000	970,000
47-4	do	447	22.4	1258	1,150,000	1,080,000	890,000
52-1	do	28	22.3	901			
52-2	do	28	22.3	914			
52-3	do	492	22.3	929	1,140,000	920,000	580,000
52-4	do	492	22.3	879	1,300,000	920,000	450,000
53-1	do	30	29.1	306			
53-3	do	30	29.1	492			
53-2	do	494	29.1	635	1,000,000	.....	.....
53-4	do	494	29.1	691	1,190,000	670,000	.....
104-5	do	32	22.9	797			
104-6	do	527	22.9	1313	1,340,000	1,280,000	1,190,000
12-1	1:1½:3 Westfield trap	28	.....	2397			
12-2	do	28	.....	1312			
12-3	do	534	.....	3880	3,520,000	3,370,000	3,270,000
12-4	do	533	.....	4180	4,390,000	4,450,000	4,130,000

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TABLE 22.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN HEAD PROTECTIONS

Cylinder No.	Kind of Concrete	Age, Days	Percent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
49-1	1:2:4 Chicago limestone	28	11 2	1473			
49-3	do	28	11. 2	1510			
49-2	do	489	11. 2	2873	8, 900, 000	6, 000, 000	6, 000, 000
49-4	do	489	11. 2	2178	3, 870, 000	3, 320, 000	3, 140, 000
59-1	do	29	10. 5	990			
59-3	do	29	10. 5	962			
59-2	do	494	10. 5	1065	2, 170, 000	1, 370, 000	830, 000
59-4	do	494	10. 5	1319	3, 780, 000	2, 870, 000	2, 250, 000
*64-1	do	27	9. 8	3428			
*64-4	do	27	9. 8	3119			
*64-2	do	502	9. 8	4036	8, 810, 000	7, 940, 000	7, 580, 000
*64-3	do	502	9. 8	4000	4, 230, 000	4, 060, 000	4, 060, 000
65-1	do	27	9. 7	2120			
65-4	do	27	9. 7	1951			
65-2	do	502	9. 7	3157	6, 500, 000	5, 990, 000	5, 830, 000
65-3	do	502	9. 7	2812	4, 300, 000	4, 050, 000	3, 770, 000
69-1	do	29	10. 9	1874			
69-3	do	29	10. 9	1698			
69-2	do	497	10. 9	2489	4, 150, 000	3, 880, 000	3, 750, 000
69-4	do	497	10. 9	2941	7, 080, 000	5, 620, 000	5, 000, 000
A-1	do	28	9. 6	2058			
A-3	do	28	9. 6	1947			
A-2	do	520	9. 6	2989	4, 040, 000	4, 020, 000	3, 810, 000
B-1	do	28	11. 2	1988			
B-3	do	28	11. 2	2197			
B-2	do	520	11. 2	3131	4, 080, 000	3, 940, 000	3, 770, 000
B-4	do	524	11. 2	2252	3, 400, 000	3, 420, 000	3, 300, 000
B-5	do	524	11. 2	2527	4, 100, 000	4, 070, 000	4, 090, 000
48-1	1:3:5 Chicago limestone	28	11. 5	961			
48-3	do	28	11. 5	1736			
48-2	do	489	11. 5	1358	2, 810, 000	2, 710, 000	2, 400, 000
48-4	do	489	11. 5	2397	3, 750, 000	3, 710, 000	3, 570, 000
62-1	do	28	11. 2	881			
62-3	do	28	11. 2	1324			
62-2	do	492	11. 2	1731	4, 580, 000	4, 260, 000	3, 730, 000
62-4	do	492	11. 2	1613	2, 900, 000	2, 770, 000	2, 370, 000
63-1	1:2:4 New York trap	27	11. 1	1408			
63-2	do	27	11. 1	1280			
63-3	do	496	11. 1	2175	2, 750, 000	2, 480, 000	2, 380, 000
63-4	do	496	11. 1	1872	3, 090, 000	2, 770, 000	2, 430, 000
*67A-1	do	28	10. 3	1945			
*67A-2	do	28	10. 3	1998			
*67A-3	do	493	10. 3	2573		7, 950, 000	6, 480, 000
*67A-4	do	493	10. 3	2406	4, 890, 000	4, 090, 000	1, 780, 000
49-5	1:2:4 Rockport granite	28	10. 6	1395			
49-7	do	28	10. 6	1509			
49-6	do	489	10. 6	2212	6, 650, 000	4, 230, 000	3, 640, 000
49-8	do	489	10. 6	2233	3, 790, 000	3, 490, 000	2, 900, 000
66-1	do	30	9. 8	1307			
66-4	do	30	9. 8	1495			
66-2	do	494	9. 8	1606	3, 580, 000	2, 960, 000	2, 570, 000
66-3	do	494	9. 8	1930	4, 080, 000	4, 000, 000	3, 500, 000
58-1	1:3:5 Rockport granite	28	13. 4	841			
58-4	do	28	13. 4	946			
58-2	do	498	13. 4	1172	2, 510, 000	2, 070, 000	1, 670, 000
58-3	do	498	13. 4	1263	2, 480, 000	2, 080, 000	1, 770, 000
105-5	1:2:4 Joliet gravel	28	10. 5	1128			
105-8	do	28	10. 5	1355			
105-6	do	538	10. 5	1921	3, 480, 000	3, 330, 000	3, 100, 000
105-7	do	538	10. 5	1723	2, 800, 000	2, 560, 000	2, 130, 000
106-5	do	29	10. 7	1396			
106-8	do	28	10. 7	1403			
106-6	do	532	10. 7	1653	3, 460, 000	3, 260, 000	3, 100, 000
106-7	do	532	10. 7	2076	3, 920, 000	3, 350, 000	3, 430, 000
105-1	1:2:4 Meramec R. gravel	28	9. 8	1721			
105-4	do	28	9. 8	1691			
105-2	do	538	9. 8	2699	5, 080, 000	4, 600, 000	4, 230, 000
105-3	do	538	9. 8	2706	3, 040, 000	3, 090, 000	3, 060, 000
109-5	do	28	8. 7	2152			
109-7	do	28	8. 7	2021			
109-6	do	500	8. 7	3232	8, 100, 000	5, 710, 000	5, 000, 000
109-8	do	500	8. 7	3095	5, 060, 000	4, 350, 000	4, 190, 000

*Time of mixing, 2 min. For all other cylinders time of mixing was 1 min.

Cylinder No. 64-2 failed on third application of maximum load, No. 64-3 on second application of maximum load.

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TABLE 23.—TESTS OF LIME

Sample No.	Quicklime		Hydrated Lime	
	1	2	1	2
Chemical Analysis:				
Impurities (SiO_2 and R_2O_3)	1.73%	1.70%	1.44%	1.31%
CaO	88.98	85.53	69.59	69.59
MgO	1.26	0.89	1.37	1.37
Loss on Ignition	8.22	8.22	27.02	27.07
CO_2 Average	0.66		4.89	
Fineness:				
Residue on No. 20 Sieve, Sample Slacked and Washed Through	1.25		....	
Residue on No. 30 Sieve, Original Sample, Dry	....		4.4	
Residue on No. 200 Sieve, Sample Washed Through	....		3.97	
Soundness: Pats—				
5 Hr. in Steam	....		Unsound	

TABLE 24.—TESTS OF CALCINED GYPSUM

	Western	Eastern
Chemical Analysis:		
Impurities (SiO_2 and R_2O_3)	3.56%	2.10%
CaO	36.77	36.93
SO_3	52.22	51.37
Loss Below 60°C.	1.23	1.12
Loss Between 60°C. and Tyrrell Burner	6.67	8.35
Time of Set:		
Neat, 55 percent Water	More than 7 hr.; less than 16 hr.	24 hr.
*Tensile Strength: Lb. per Sq. In.		
Neat, 55 percent Water	200.0 190.0 192.0	Easily crumbles No test obtainable
1 Gypsum: 3 Ottawa Sand, 22 percent Water	97.0 102.0 85.0	175.0 150.0 150.0

*Mortars made up with distilled water and after setting 24 hr. in air, were dried to constant weight below 60°C. and then broken.

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TABLE 25.—COMPRESSIVE STRENGTH OF PORTLAND CEMENT AND LIME PLASTER

Test specimens 2-in. cubes stored in air

Cube No.	Proportion, Parts by Loose Volume of Materials as Used	Percent Water	Age, Days	Compressive Strength, Lb. per Sq. In.
22-1	1 Portland cement			
	1/10 Hydrated lime	15.78	28	1650
	2½ Coarse lake sand			
22-2	do	15.78	28	1455
22-3	do	15.78	512	1823
22-4	do	18.23	29	1488
22-5	do	18.10	29	1795
22-7	do	18.10	511	2805
22-8	do	15.23	29	2715
22-9	do	15.23	509	3628
24-1	do	16.23	28	1530
24-2	do	16.23	501	2855
24-3	do	17.79	29	1360
24-4	do	17.79	486	2140
24-5	do	12.87	29	933
24-6	do	12.87	486	1689
25-1	do	16.20	28	2530
25-2	do	16.20	487	3956
25-3	do	18.50	29	1319
25-4	do	18.50	484	2480
25-1	do	18.35	29	1655
25-2	do	18.35	500	3085
25-3	do	15.82	28	2685
25-4	do	15.82	496	3744
27-1	do	18.00	29	1645
27-2	do	18.00	501	3191
27-3	do	16.50	29	976
27-4	do	16.50	499	1984
78-1	do	19.52	38	818
78-2	do	19.52	38	795
78-3	do	19.52	38	1038
78-4	do	19.52	38	640
78-5	do	19.52	38	610
78-6	do	19.52	38	790
78-7	do	21.15	34	991
78-8	do	21.15	34	583
78-9	do	21.15	34	1035
110-1	do	21.60	28	1455
110-2	do	21.60	28	1200
110-3	do	21.60	28	1225
110-4	do	21.60	514	1805
110-5	do	21.60	514	1595
110-6	do	21.60	514	1946
110-7	do	17.20	28	1913
110-8	do	17.20	28	1748
110-9	do	17.20	28	1716
110-10	do	17.20	512	2181
110-11	do	17.20	512	3408
110-12	do	17.20	512	2639
110-13	do	19.00	28	1789
110-14	do	19.00	28	1863
110-15	do	19.00	28	1831
110-16	do	19.00	507	1980
110-17	do	19.00	507	2990
110-18	do	19.00	507	2753
110-19	do	19.02	29	2084
110-20	do	19.02	29	2020
110-21	do	19.02	29	1710
110-22	do	19.02	500	2838
110-23	do	19.02	500	2758
110-24	do	19.02	500	3248
77-4	1 Slaked lime			
	2½ Fine lake sand	18.36	60	158
77-5	do	18.36	60	149

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TABLE 26.—COMPRESSIVE STRENGTH OF CLAY TILE MORTAR

Test specimens 2-in. cubes stored in air

Cube No.	Proportion, Parts by Loose Volume of Materials as Used	Percent Water	Age, Days	Compressive Strength, Lb. per Sq. In.
	1 Portland cement			
48-1	1 Slaked lime	24.00	28	955
	4 Bank or beach sand			
48-2	do	24.00	28	873
48-3	do	24.00	499	1040
48-4	do	24.00	499	769
48-5	do	27.50	28	768
48-6	do	27.50	512	833
49-1	do	25.00	28	421
49-2	do	25.00	498	485
49-3	do	27.15	28	410
49-4	do	27.15	498	405
50-1	do	26.00	28	273
50-2	do	26.00	496	361
51-1	do	25.70	28	385
51-2	do	25.70	497	358
51-3	do	28.45	28	278
51-4	do	28.45	497	308
51A-1	do	24.45	29	303
51A-2	do	24.45	516	313
53-1	do	24.40	28	445
53-2	do	24.40	503	370
56-1	do	24.60	28	455
56-2	do	24.60	28	399
56-3	do	24.60	500	565
56-4	do	24.60	500	550
57-1	do	22.70	29	513
57-2	do	22.70	497	700
58-1	do	24.80	28	288
58-2	do	24.80	504	373
58-3	do	24.10	28	340
58-4	do	24.10	504	348
58-5	do	28.00	28	283
58-6	do	28.00	504	325
59-1	do	25.90	28	395
59-2	do	25.90	498	333
59-3	do	24.75	28	368
59-4	do	24.75	497	270
60-1	do	25.00	28	296
60-2	do	25.00	505	285
60-3	do	26.60	28	360
60-4	do	26.60	505	358
61-1	do	28.05	29	513
61-2	do	28.05	29	390
61-3	do	28.05	524	473
61-4	do	28.05	524	628
61-5	do	23.50	29	415
61-6	do	23.50	524	486
62-1	do	25.30	28	475
62-2	do	25.30	483	551
63-1	do	28.25	28	318
63-2	do	28.25	505	350
68-1	do	20.50	28	310
68-2	do	20.50	503	270
68-3	do	24.10	28	285
68-4	do	24.10	503	323
69-1	do	21.80	516	463
69-3	do	24.45	29	318
69-4	do	24.45	516	338
69-5	do	21.45	28	853
69-6	do	21.45	515	940
69-7	do	22.65	28	653
69-8	do	22.65	28	663
69-9	do	22.65	493	773

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TABLE 26.—COMPRESSIVE STRENGTH OF CLAY TILE MORTAR—Concluded

Test specimens 2-in. cubes stored in air

Cube No.	Proportion, Parts by Loose Volume of Materials as Used	Percent Water	Age, Days	Compressive Strength, Lb. per Sq. In.
69–10	1 Portland cement 1 Slaked lime 4 Bank or beach sand	22.65	498	708
76–1	do	23.75	60	495
76–2	do	23.75	60	571
76–3	do	23.75	60	583
77–1	do	23.75	60	330
77–2	do	23.75	60	385
77–3	do	23.75	60	360
52–2	1 Portland cement 2 Slaked lime 6 Bank sand	29.40	36	228
52–5	do	30.30	29	100
54–5	do	26.60	28	159
54–6	do	26.60	498	216
54–1	1 Portland cement 1½ Hydrated lime 6 Bank sand	29.90	28	303
54–3	do	30.90	28	395
54–4	do	30.90	499	636

TABLE 29.—COMPRESSIVE STRENGTH OF GYPSUM FILLING

Test specimens 8 by 16 in. cylinders

Cylinder No.	MATERIAL	Proportions, Parts by Loose Volume of Materials as Used	*Percent Water	Age, Days	Ultimate Strength, Lb. per Sq. In.
66–1	Eastern gypsum filling	1 Eastern calcined gypsum 1 Fine lake sand 4 Broken gypsum blocks	54.70	31	15
66–5	do	do	61.20	29	21
66–6	do	do	61.20	483	95
67A–5	do	do	60.15	29	22
67A–6	do	do	60.15	29	17
67A–7	do	do	60.15	489	105
67A–8	do	do	60.15	489	85
109–3	Specimen collapsed on removal	of mould 12 days after placement	52.20	.....	.....
109–4	Broken during storage	.....	52.20	.....	.....
108–1	Western gypsum filling	1 Western calcined gypsum 1 Fine lake sand 4 Broken gypsum blocks	63.30	28	27
108–2	do	do	63.30	28	32
108–3	do	do	63.30	513	42
108–4	do	do	63.30	513	32
109–1	do	do	63.10	28	29
109–2	do	do	63.10	509	40

* Based on total weight of dry materials in mixture.

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TABLE 27.—COMPRESSIVE STRENGTH OF GYPSUM MORTAR AND PLASTER

Test specimens 2-in. cubes stored in air

Cube No.	Proportion, Parts by Loose Volume of Materials as Used	*Percent Water	Age, Days	Compressive Strength, Lb. per Sq. In.
64–1	1 Western calcined gypsum			
	3 Fine lake sand	23.40	30	220
64–3	do	23.40	30	145
64–4	do	23.40	523	118
64–5	do	23.40	523	94
65–1	do	22.20	29	283
65–2	do	22.20	29	275
65–3	do	22.20	29	275
65–4	do	22.20	522	202
65–5	do	22.20	522	214
66–1	1 Eastern calcined gypsum			
	3 Fine lake sand	23.50	29	43
66–2	do	23.50	29	13
66–3	do	23.50	29	15
66–5	do	23.50	504	35
67–1	do	23.10	28	193
67–2	do	23.10	28	105
67–3	do	23.10	28	213
67–5	do	23.10	520	152
67–6	do	23.10	503	160
67A–1	do	22.85	31	215
66A–2	do	22.85	31	130
67A–3	do	22.85	31	188
67A–6	do	22.85	497	83
108–1	do	22.60	27	189
108–2	do	22.60	27	201
108–3	do	22.60	489	83
108–4	do	20.60	27	240
108–5	do	20.60	489	139
108–5	do	20.60	489	172
109–1	do	22.20	28	70
109–2	do	22.20	28	86
109–3	do	22.20	515	75
109–4	do	22.60	515	139
109–5	do	22.60	28	200
109–6	do	22.60	515	222
76–4	1 Fibered gypsum plaster			
	3 Fine lake sand	17.20	60	443
76–5	do	17.20	60	468
76–6	1 Calcined gypsum			
	2 Hydrated lime	61.00	60	958
77–6	do	61.00	60	1029

*Based on total weight of dry materials in mixture.

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TABLE 28.—STRENGTH TESTS OF MORTAR AND PLASTER

Test specimens, 1-in. briquettes for tension tests and 2-in. cubes for compression tests made in laboratory. Each value is the average of three tests.

Mortar or Plaster	Proportion, Parts by Loose Volume of Materials as Used	*Percent Water	Method of Storage	Age, Days	Strength, Lb. per Sq. In.	
					Tensile	Compressive
Clay tile mortar	1 Portland cement 1 Slaked lime 4 Bank sand	25.0	1 day in damp closet, remainder of period in water	7	67	257
do	do with beach sand	25.0	do	7	72	236
do	do with Ottawa sand	25.0	do	7	65	174
do	do with bank sand	25.0	do	28	97	425
do	do with beach sand	25.0	do	28	114	406
do	do with Ottawa sand	25.0	do	28	91	369
do	do with Ottawa sand		do	28	161	848
do	do with bank sand	25.0	1 day in damp closet, remainder of period in air	28	.....	328
do	do with Ottawa sand	25.0	do	28	.....	190
do	do with bank sand	25.0	do	365	.....	442
do	do with Ottawa sand	25.0	do	365	.....	259
Portland cement plaster	1 Portland cement 1/10 Hydrated lime 2½ Coarse lake sand	16.7	1 day in damp closet, remainder of period in water	7	144	418
do	do with Ottawa sand	16.7	do	7	114	363
do	do with coarse lake sand	16.7	do	28	197	1105
do	do with Ottawa sand	16.7	do	28	154	721
do	do with coarse lake sand	16.7	1 day in damp closet, remainder of period in air	28	.....	1010
do	do with Ottawa sand	16.7	do	28	.....	616
do	do with coarse lake sand	16.7	do	365	.....	1297
do	do with Ottawa sand	16.7	do	365	.....	963
Western gypsum block mortar	1 Western gypsum 3 Fine lake sand	22.0	1 day in air, then dried to constant weight below 60° C.	2	126	.....
do	do with Ottawa sand	22.0	do	2	95	.....
Eastern gypsum block mortar	1 Eastern gypsum 3 Fine lake sand	22.5	do	2	202	.....
do	do with Ottawa sand	22.5	do	2	158	.....
Western gypsum block mortar	1 Western gypsum 3 Fine lake sand	22.0	In air for whole period.	28	.....	491
do	do with Ottawa sand	22.0	do	28	.....	343
do	do with fine lake sand	22.0	do	365	.....	647
do	do with Ottawa sand	22.0	do	365	.....	517
Eastern gypsum block mortar	1 Eastern gypsum 3 Fine lake sand	22.5	do	28	.....	786
do	do with Ottawa sand	22.5	do	28	.....	415
do	do with fine lake sand	22.5	do	365	.....	1180
do	do with Ottawa sand	22.5	do	365	.....	698

*Based on total weight of dry materials in mixture.

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TABLE 30.—CLASSIFICATION AND DESCRIPTION OF HOLLOW CLAY TILE

Tile No.	Nominal Thickness, In.	Type of Clay	District	Hardness	Color	Average Outside Dimensions, In.			Thickness of Side Shells, In.	Cross Webs		Scoring		
						Thickness	Width	Length		Number	Av. Thickness, In.	Number	Shape	Dimensions, In.
A	2	Surface clay....	Chicago, Ill.....	Soft.....	Light red.	2.06	12.00	12.00	0.80	4	0.80	22	Rectangular	0.12 by 0.50
A	4	do.....	do.....	Soft.....	Light red.	4.09	12.20	12.00	0.79	4	0.71	22	do.....	0.12 by 0.50
B	2	do.....	Boston, Mass.....	Medium	Red.....	2.10	6.10	12.00	0.65	3	0.68	12	Half round..	0.30 diam.
B	4	do.....	do.....	Medium	Red.....	4.03	12.30	11.70	0.83	5	0.75	24	do.....	0.40 diam.
C	2	Semi-fire clay..	Waynesburg, O...	Hard....	Buff.....	2.07	12.02	11.90	0.55	4	0.50	24	Rectangular	0.12 by 0.45
C	4	do.....	do.....	Hard....	Buff.....	3.98	11.84	11.85	0.53	4	0.46	28	do.....	0.12 by 0.45
D	2	Shale.....	Cleveland, O.....	Hard....	Red.....	2.01	11.90	11.65	0.52	4	0.50	26	do.....	0.02 by 0.25
D	4	do.....	do.....	Hard....	Red.....	3.96	11.62	11.85	0.60	4	0.56	40	do.....	0.10 by 0.30
E	2	Semi-fire clay..	Perth Amboy, N. J	Medium	Buff.....	2.09	7.96	12.20	0.70	4	0.82	22	Half round..	0.20 diam.
E	4	do.....	do.....	Medium	Buff.....	3.85	11.87	11.87	0.67	4	0.64	38	do.....	0.30 diam.
F	2-in. curved	do.....	do.....	Soft.....	Light red.	2.09	10.56	12.00	0.85	4	0.96	11	do.....	0.30 diam.
G	2-in. curved	do.....	do.....	Soft.....	Light red.	2.02	14.00	12.00	0.83	4	1.45	16	do.....	0.35 diam.
H	2-in. curved	do.....	do.....	Soft.....	Light red.	2.55	10.94	12.00	0.76	4	0.67	11	Rectangular	0.10 by 0.37
												3	do.....	0.40 by 2.74

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TABLE 31.—COMPRESSIVE STRENGTH OF HOLLOW CLAY TILE

Tile units are 12 in. wide and 12 in. long, except as noted

Sample No.	Kind of Title		Porosity, Percent of Volume	Absorption, Percent of Dry Weight	How Tested	Area under Load, Sq. In.	Maximum Load	
	Nominal Thickness, In.	Clay					Total, Lb.	Lb. per Sq. In.
A-1	2	Surface clay,	40.3	29.0	On end	19.6	44300	2260
A-2	2	Chicago	44.4	28.5	do	19.6	46860	2390
A-3	2	district	43.1	27.3	do	19.6	44650	2280
Average			42.6	28.3			45277	2310
A-4	2	do	44.5	28.5	On edge	16.2	18920	1170
A-5	2	do	43.7	27.7	do	16.2	32110	1980
A-6	2	do	41.9	26.2	do	16.2	22340	1380
Average			43.4	27.5			24456	1510
A-16	4	do			On end	25.2	53000	2100
A-19	4	do	34.4	27.6	do	25.2	57280	2270
A-20	4	do			do	25.2	63460	2520
Average							57913	2296
A-11	4	do	44.2	28.1	On edge	16.0	29320	1830
A-12	4	do	42.9	29.5	do	16.0	20250	1270
A-14	4	do	41.8	26.6	do	16.0	33190	2070
Average			42.9	27.5			27588	1723
*B-6	2	Surface clay,	28.8	15.0	On end	9.1	52830	5800
*B-8	2	Boston	27.3	15.0	do	9.1	48400	5310
*B-10	2	district	27.9	14.2	do	9.1	62450	6800
Average			28.0	14.7			54560	5990
*B-1	2	do	30.0	16.0	On edge	11.7	52450	4480
*B-2	2	do	28.6	15.0	do	11.7	58150	4970
*B-4	2	do	31.0	16.6	do	11.7	58040	4900
Average			29.8	15.8			56213	4796
B-12	4	do	30.8	16.4	On end	28.4	134350	4740
B-14	4	do	32.4	17.6	do	28.4	111400	3920
B-19	4	do	25.6	12.8	do	27.2	160930	5910
Average			29.6	15.6			135560	4856
B-11	4	do	28.8	15.6	On edge	15.3	68170	4460
B-13	4	do	28.0	14.6	do	15.3	69760	4560
B-17	4	do	29.8	15.6	do	14.1	56710	4020
Average			28.8	15.1			64880	4346
C-1	2	Ohio semi-fire clay	15.5	7.3	On end	14.1	98350	6980
C-5	2	do	15.4	7.0	do	14.1	123500	8770
C-6	2	do	22.5	11.2	do	14.7	94280	6410
Average			17.8	8.5			105376	7386
C-2	2	do	18.3	8.8	On edge	11.2	68870	6150
C-4	2	do	18.5	8.8	do	11.2	47040	4200
C-8	2	do	16.9	7.9	do	11.2	46920	4190
Average			17.9	8.8			54276	4846
C-13	4	do	18.5	9.0	On end	16.7	123610	7400
C-14	4	do	17.6	8.6	do	17.2	104850	6090
C-19	4	do	18.5	8.9	do	16.7	96050	5760
Average			18.2	8.8			108170	6416
C-15	4	do	16.0	8.4	On edge	9.6	58180	6060
C-18	4	do			do	10.0	36780	3680
Average							47480	4870

*Nominal width of tile, 6 in.

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TABLE 31.—COMPRESSIVE STRENGTH OF HOLLOW CLAY TILE—Concluded

Tile units are 12 in. wide and 12 in. long, except as noted

Sample No.	Kind of Tile		Porosity, Percent of Volume	Absorption, Percent of Dry Weight	How Tested	Area under Load, Sq. In.	Maximum Load	
	Nominal Thickness, In.	Clay					Total, Lb.	Lb. per Sq. In.
D-1	2	Ohio shale	8.1	3.5	On end	14.3	114350	8000
D-3	2	do	22.1	10.8	do	14.1	112600	7990
D-9	2	do	14.3	7.2	do	14.5	109680	7560
Average			14.8	7.1			112210	7850
D-2	2	do	16.2	7.4	On edge	11.1	43200	3900
D-5	2	do	12.2	5.5	do	10.7	61980	5800
D-8	2	do	8.2	3.6	do	11.1	81450	7340
Average			12.2	5.5			62243	5680
D-11	4	do	12.0	5.2	On end	19.9	193550	9720
D-16	4	do	11.3	4.9	do	19.1	151770	7950
D-17	4	do	12.2	5.5	do	19.9	177940	8930
Average			11.8	5.2			174420	8866
D-13	4	do	16.3	7.4	On edge	11.7	95840	8190
D-14	4	do	13.8	6.5	do	11.7	61130	5220
D-19	4	do	9.6	3.8	do	11.5	117790	10230
Average			13.2	5.9			91666	7880
*E-5	2	Semi-fire clay, New Jersey district	28.3	17.2	On end	12.1	53240	4400
*E-6	2		30.4	17.1	do	12.1	51940	4210
*E-8	2				do	12.1	55620	4600
Average							53313	4402
*E-1	2	do	33.2	19.8	On edge	15.6	33670	2160
*E-2	2	do	33.5	19.6	do	15.9	29000	1820
*E-10	2	do	35.8	22.0	do	15.9	23100	1450
Average			34.1	20.4			28590	1810
E-15	4	do	25.3	14.2	On end	21.6	95650	4440
E-17	4	do	33.6	20.6	do	21.6	74660	3460
E-19	4	do	29.5	17.2	do	21.6	86250	4000
Average			29.4	17.3			85520	3966
E-12	4	do	29.0	16.2	On edge	12.6	39900	3170
E-13	4	do	33.2	19.8	do	12.6	24920	1980
E-14	4	do	33.9	20.4	do	12.6	20050	1590
Average			32.0	15.4			28290	2246
†F-5	2	Porous semi-fire clay, New Jersey district	37.6	23.2	On end	18.9	83270	4410
†F-8	2		41.8	27.0	do	19.9	54200	2720
†F-10	2		45.3	30.4	do	18.3	72630	3970
Average			41.5	23.8			70033	3700
†G-2	2	do	42.1	27.6	do	24.6	38060	1550
†G-3	2	do	50.2	39.1	do	23.9	33370	1400
†G-4	2	do	46.2	26.4	do	23.9	53440	2230
Average			46.5	31.0			41623	1726
†H-1	2½	do	51.2	39.0	do	19.9	27150	1360
†H-2	2½	do	46.9	34.3	do	18.0	32110	1780
†H-4	2½	do	41.4	27.7	do	20.8	40300	1940
Average			46.5	33.3			33086	1693

*Nominal width of tile, 8 in. †Curved tile.

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TABLE 32.—TRANSVERSE STRENGTH OF HOLLOW TILE

Tile tested flatwise on side with center load and supports 10 in. apart. Tile units are 12 in. wide, except as noted.

Sample No.	Kind of Tile		Maximum Load, Lb.	Calculated Maximum Stresses, Lb. per Sq. In.	
	Nominal Thickness, In.	Clay		Bending, Outer Fiber	Shear, Center of Webs
A-7	2	Surface clay, Chicago district	1120	365	158
A-8	2	do	1050	370	148
A-9	2	do	830	292	117
A-10	2	do	1100	387	155
Average			1025	361	145
A-13	4	do	2600	254	146
A-15	4	do	1500	147	84
A-17	4	do	2500	244	140
A-18	4	do	1890	185	105
Average			2122	207	119
*B-3	2	Surface clay, Boston district	1000	625	202
*B-5	2	do	1520	950	307
*B-7	2	do	1280	800	258
*B-9	2	do	1240	775	250
Average			1260	787	254
B-15	4	do	5600	547	253
B-16	4	do	6900	675	312
B-18	4	do	5100	499	231
B-20	4	do	5360	513	237
Average			5712	558	283
C-7	2	Ohio semi-fire clay	1220	477	236
C-10	2	do	2040	798	394
Average			1630	637	315
C-20	4	do	2120	321	223
D-4	2	Ohio shale	2530	945	415
D-6	2	do	1630	608	267
D-7	2	do	3200	1194	525
Average			2482	916	402
D-12	4	do	4530	364	320
D-15	4	do	6250	778	441
D-18	4	do	4910	611	346
D-20	4	do	3280	408	231
Average			4742	590	335
†E-3	2	New Jersey semi-fire clay	1410	705	155
†E-4	2	do	1100	550	121
Average			1255	627	138
E-11	4	do	2010	272	142
E-16	4	do	1900	257	134
E-18	4	do	1330	180	94
E-20	4	do	2570	347	182
Average			1962	264	140

*Nominal width, 6 in. †Nominal width, 8 in.

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TABLE 33.—TEMPERATURES OF VITRIFICATION AND FUSION OF CLAY TILE AND BRICK

No.	CLAY	Temperature of Vitrification, Deg. C.	Temperature Producing Softening, Deg. C.	Temperature of Fusion, Deg. C.
A	Surface clay, Chicago district	1120	1200	1240
B	Surface clay, Boston district	1120	1160	1180
C	Ohio semi-fire clay	1180	1450	1460
D	Ohio shale	1120	1380	1400
E	New Jersey semi-fire clay partition tile	1200	1480	1500
F	New Jersey porous semi-fire clay, 2 by 8-in. curved tile	....	1450	1470
G	New Jersey porous semi-fire clay, 2 by 12-in. curved tile	....	1510	1530
H	New Jersey porous semi-fire clay, 2½ by 11-in. curved tile	....	1450	1470
J	Chicago common brick	....	....	1145

TABLE 34.—POROSITY AND ABSORPTION OF CHICAGO COMMON BRICK

Brick No.	Specimen No.	Porosity, Percent of Volume	Absorption, Percent of Dry Weight
1	1	35.4	20.2
1	2	31.9	17.8
1	3	32.5	18.0
1	4	33.5	17.6
1	5	24.5	13.1
1	6	36.3	20.9
1	7	26.5	15.4
1	8	26.3	12.9
1	Average	30.4	16.9
2	1	43.1	26.8
2	2	42.1	26.9
2	3	40.1	24.4
2	4	39.9	24.4
2	5	40.3	24.7
2	6	40.9	25.1
2	7	40.3	24.6
2	8	40.6	24.9
2	Average	40.9	25.1

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TABLE 35.—COMPRESSIVE STRENGTH OF CHICAGO COMMON BRICK

Sample No.	How Tested	Dimensions, In.			Area Under Load, Sq. In.	Maximum Load	
		Width	Thickness	Length		Total, Lb.	Lb. per Sq. In.
J-1	On end	3.45	2.20	....	7.59	32800	4330
J-2	do	3.75	2.25	....	8.44	14800	1760
J-4	do	3.50	2.25	....	7.88	27550	3500
Average							3200
J-3 ½ brick	On edge	....	2.25	2.9	6.52	7550	1160
J-5 do	do	....	2.15	3.35	7.20	26000	3610
J-7 do	do	....	2.35	5.49	12.69	15680	1240
J-8 do	do	....	2.35	4.10	9.64	16800	1740
J-9 do	do	....	2.25	3.40	7.67	15150	1980
J-10 do	do	....	2.25	4.55	10.24	20700	2020
Average							1960
J-3 ½ brick	On side	3.70	....	4.30	15.17	46010	3080
J-5 do	do	3.50	....	4.60	16.10	55800	3460
J-6 do	do	3.60	....	7.80	28.08	57730	2050
J-7 do	do	3.50	....	2.60	9.10	16490	1810
J-9 do	do	3.60	....	4.60	16.56	67910	4100
J-10 do	do	3.65	....	3.70	13.50	32900	2440
Average							2815

TABLE 36.—TRANSVERSE STRENGTH OF CHICAGO COMMON BRICK

Brick tested on side with center load and supports 7 in. apart

Sample No.	Dimensions, In.		Maximum Load, Lb.	Modulus of Rupture Lb. per Sq. In.
	Width	Thickness		
J-3	3.70	2.25	940	530
J-5	3.50	2.15	2700	1750
J-8	3.70	2.35	460	240
J-9	3.60	2.25	1540	890
J-10	3.65	2.25	1590	900
Average				882

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TABLE 37.—POROSITY OF GYPSUM BLOCK

Sample No.	Kind of Block		Porosity, Percent of Total Volume
	Thickness	Gypsum	
K-4	4-in. hollow	Western	63.7
K-5	4-in. hollow	do	63.7
L-4	4-in. solid	do	63.7
L-5	4-in. solid	do	63.9
M-4	2-in. solid	do	63.6
M-5	2-in. solid	do	63.6
Average			63.7
N-4	4-in. solid	Eastern	58.6
N-5	4-in. solid	do	58.4
O-4	2-in. solid	do	62.6
O-5	2-in. solid	do	62.2
Average			60.4

TABLE 38.—COMPRESSIVE STRENGTH OF SOLID GYPSUM BLOCK

All blocks tested on edge

Sample No.	Gypsum	Dimensions, In.			Area Under Load, Sq. In.	Maximum Load	
		Thickness	Length	Width		Total, Lb.	Lb. per Sq. In.
L-1	Western	4	30.1	12	120.5	61100	507
L-2	do	4	30.1	12	120.5	44300	367
L-3	do	4	30.0	12	120.0	49080	409
Average							428
M-1	do	2	29.9	12	59.8	38350	640
M-2	do	2	29.9	12	59.8	23760	396
M-3	do	2	30.0	12	60.0	23840	398
Average							478
N-1	Eastern	4	25.7	16.5	103.0	67540	656
N-2	do	4	25.7	16.5	103.0	51240	498
N-3	do	4	26.2	16.5	105.0	52960	504
Average							553
O-1	do	2	26.1	16.5	52.2	23960	458
O-2	do	2	26.1	16.5	52.2	19205	365
O-3	do	2	26.1	16.5	52.2	21685	415
Average							413

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TABLE 39.—TRANSVERSE STRENGTH OF SOLID GYPSUM BLOCK

All blocks tested flatwise on side with center load

Sample No.	Gypsum	Dimensions, In.			Span, In.	Weight of Block, Lb.	Maximum Load, Lb.	Modulus of Rupture, Lb. per Sq. In.
		Thickness	Length	Width				
L-7	Western	4	30.2	12	24	42.9	689	132
L-8	do	4	30.2	12	24	47.4	652	125
L-9	do	4	30.2	12	24	46.2	562	109
Average								122
M-7	do	2	29.8	12	24	22.7	266	206
M-8	do	2	29.9	12	24	22.1	260	201
M-9	do	2	29.9	12	24	22.8	294	227
Average								211
N-7	Eastern	4	26.1	16.5	20	58.9	1520	175
N-8	do	4	26.1	16.6	20	56.2	1428	165
N-9	do	4	26.0	16.6	20	58.2	1568	169
Average								170
O-7	do	2	25.9	16.4	20	24.4	305	142
O-8	do	2	26.2	16.3	20	27.7	245	117
O-9	do	2	25.9	16.4	20	25.7	311	146
Average								135

TABLE 40.—TRANSVERSE STRENGTH OF GYPSUM WALL BOARD

Samples, 18 in. long, were tested with center load and supports 16 in. apart

No.	Width, In.	Thickness, In.	Weight, Lb.	Condition of Board	Direction of Loading Bar	Maximum Load, Lb.
A-1	11.9	0.38	3.1	Dry	Parallel with grain of paper	37.0
A-2	12.0	0.39	3.1	Dry	do	39.0
A-3	12.0	0.39	3.1	Dry	do	37.5
Average						37.8
B-1	12.0	0.39	3.1	Dry after having been saturated	Parallel with grain of paper	39.0
B-2	11.9	0.38	3.0		do	35.5
Average						37.2
C-1	11.9	0.40	3.8	Saturated	Parallel with grain of paper	10.0
C-2	12.0	0.38	3.7	Saturated	do	6.0
Average						8.0
D-1	12.0	0.38	3.1	Dry	Perpendicular to grain of paper	90.5
D-2	12.0	0.34	3.2	Dry	do	98.0
D-3	12.0	0.39	3.2	Dry	do	92.0
Average						92.8
E-1	12.0	0.38	3.0	Dry after having been saturated	Perpendicular to grain of paper	89.5
E-2	12.0	0.38	3.1			87.0
Average						88.2
F-1	12.0	0.38	3.7	Saturated	Perpendicular to grain of paper	15.5

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APPENDIX E.—PREVIOUS INVESTIGATIONS

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The first experimental investigations on the fire resistance of building columns were made abroad, principally in Germany. When iron came into use as a structural material in that country, it was thought that the possibility of constructing truly fireproof buildings was realized, for iron is not combustible. Extensive fires showed, however, that, while the structural framework did not burn, the building collapsed suddenly and without warning. It was early recognized that some wooden structures offered greater resistance to fire than those built of unprotected iron, a result that occasioned no little comment when first observed. In fact, iron as a structural material was for a time in considerable disrepute. At one time the use of unprotected cast-iron columns under main bearing walls was forbidden in Berlin but wrought-iron columns permitted. Subsequent to large fires in Berlin and Hamburg the reverse was true. Similar changes in opinion were evident as to whether cast and wrought iron columns should be given fire-protective coverings and whether such coverings should be removable or permanent.

1. BAUSCHINGER'S TESTS[8]

In 1884–1886 Prof. J. Bauschinger, of Munich, Germany, made two series of fire and water tests on building columns that were loaded in a horizontal testing machine and heated by wood fire in a wrought-iron trough placed under them. Water was applied to the top surface. Temperatures were measured by alloys of tin, lead, and silver, having computed melting points of 300, 400, 500, and 600° C, that were attached to rods and held against the surface of the column at the middle of the sides to obtain the average temperature. Deflections were measured in the vertical and horizontal directions by indicators attached to wires that were fastened to the column at the middle of its length. The columns were loaded to what were considered safe working loads and subjected to three successive fire and water tests, the surface temperatures it was aimed to attain when water was applied being generally 300° C, 400 to 600° C, and red heat above 600° C. These tests were made with the column ends fixed or restrained. If failure did not occur, a final test at red heat with the column ends unrestrained was made in most cases. The method of testing was largely determined by the fact that opinions differed as to whether damage to cast-iron columns in building fires was caused by the fire or by the application of water to the red-hot metal.

During the first part of each fire test the column deflected downward toward the fire, due to the unequal heating, but with increase of temperature greater uniformity obtained and the column straightened somewhat. The application of water to the upper surface caused another sharp deflection downward, which became less as the

[8] Mittheilungen aus den Mech. Tech. Lab. d. k. Tech. Hochschule, München, Heft 12, 1885; Heft 15, 1887.

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column cooled on all sides, the final deflection being in some cases upward. The test conditions and effects approximate to some extent those pertaining to unprotected columns under exterior walls.

(*a*) FIRST SERIES.—In the first series tests were made on six cast-iron, three wrought-iron, and 15 columns of other building materials, including Portland-cement mortar, brick, and several kinds of building stone.

The cast-iron columns had been rejected for building purposes because of uneven wall thickness, "cold shuts," and other defects, and were more or less ornamental in form, the types varying from the plain cylindrical and slightly tapering shafts to the form having an ornamental base for about one-third of the length and a tapering shaft, which in one case was deeply fluted. They had plain or highly ornamental capitals and were from 11 feet to 13.8 feet in length and from 5.8 to 7.6 inches in outside diameter, as measured at the midheight of the column, with average wall thicknesses of 0.40 to 1 inch, the thickness varying considerably within each column. Working loads of 3400 to 5800 pounds per square inch were applied, the stress depending on the slenderness ratio of the column.

As tested with restrained ends the cast-iron columns supported their full load in the fire and water tests, although cracks developed in some cases at the higher temperatures when water was applied. Maximum vertical deflections of 3½ inches were observed after application of water on columns of nearly uniform wall thickness. With columns of uneven wall thickness the deflections were larger. In the tests with unrestrained ends, on application of water the resulting deflection made the columns unable to support full load and caused some of them to break.

The wrought-iron columns were about 13 feet (4 meters) in length and consisted of one welded tube 5.04 inches outside diameter and 0.24 inch thick, and two built-up box columns, one of two 7-inch channels and two plates fastened together by bolts spaced about 16 inches on centers, and the other of two 7-inch I beams and two plates fastened with bolts spaced about 7 inches apart. The loads were 6200, 7600, and 6700 pounds per square inch, respectively.

The welded tube took a large deflection before the temperature had reached 600° C and failed to sustain full load. A slight application of water increased the deflection and caused it to fall out of the machine. The two other columns were heated to 300 and 400° C, followed in each case by water application, their behavior being similar to that of the cast-iron columns, only the deflections were larger. On heating to 600° C and applying water the deflections became so large that the full load could not be carried, and some of the bolts were sheared off.

Of the tests on other building materials that with Portland-cement mortar was made on a column about 12 inches (30 cm) square and 10 feet (3 m) long. The proportions of the mixture was 1:5 Portland cement and coarse sand. The column was tested at the age of 6½ months under a working load of 95 pounds per square inch. It was heated for 1¾ hours until a temperature of 600° C was attained at the middle of the sides, when water was applied. No apparent injury resulted, and when cold it was loaded to failure at 920 pounds per square inch.

The brick column was about 12½ inches (32 cm) square and 6.6 feet long with brick laid in Portland-cement mortar and the outside covered with a layer of Roman-cement plaster about 0.60 inch (1.5 cm) thick. It was exposed to fire for one hour attaining a temperature of 600° C on the sides, when water was applied. No apparent damage other than slight cracking and flaking of the plaster resulted, the column being subsequently loaded to failure at load of 520 pounds per square inch.

(*b*) SECOND SERIES.—Objections raised against Bauschinger's tests, in particular that the loads applied to the cast iron-columns were too small and that the component

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parts of the wrought-iron columns were bolted together at intervals too far apart to secure a rigid section led Bauschinger to conduct a second series of tests in 1886 on two cast and five wrought iron columns. The cast-iron columns were about 13 feet (4 m) long; one had outside diameter of 7 inches and average wall thickness of 1.1 inch, and the other had outside diameter of 6.1 inch and average wall thickness of 1 inch, the applied working loads being 8400 and 7100 pounds per square inch, respectively. The columns had fairly uniform wall thickness and were apparently of better quality than the cast-iron columns of the first series. They were tested in a manner similar to those in the first series, and although large deflections occurred when heated to red heat and suddenly cooled by water application, they sustained their full working load. No cracks due to water application developed.

The wrought-iron columns were about 19 feet long and consisted of two plate and channel box sections and three starred-angle sections. The former were built up of two 5.7-inch channels and two $\frac{1}{16}$ by 8 inch (20 cm) plates with rivets spaced 3 to 5 inches on centers, and were loaded to 6300 pounds per square inch. The starred-angle sections were of two sizes, two being built up of four $3\frac{1}{2}$ by $3\frac{1}{2}$ by $\frac{7}{16}$ inch angles spaced $2\frac{1}{4}$ inches back to back and riveted together rigidly at the ends and at two intermediate points about 6 feet apart. The angles of the third column of this type had $3\frac{1}{2}$ by $3\frac{1}{2}$ by $\frac{5}{16}$ inch (1 cm) angles spaced 1.4 inches back to back and were riveted at the ends and at five intermediate points about 3 feet apart. The working loads for the two types were 6300 and 5300 pounds per square inch, respectively.

All five columns were heated red hot, and water applied until they were cold. The two box columns carried their full load throughout the test, although large permanent deflections were produced. The angle columns deflected downward rapidly and were unable to carry their full load at 600° C. Water application increased the deflection and caused a further decrease in the sustained load. The rivets were distorted, but none were sheared off

2. TESTS BY MÖLLER AND LÜHMANN[4]

In 1887, M. Möller, a Government architect, and R Lühmann, an engineer, obtained a prize offered by the (German) Society for the Promotion of Industrial Progress for the best essay on the subject of the "Resistance of Iron Columns when Subjected to High Temperatures," submitting to that end a paper in which were described tests conducted by them at Hamburg, Germany.

From the remarks accompanying the announcement of the competition it is evident that the principal interest centered in the relative resistance to fire and water of unprotected cast and wrought iron columns, although the behavior of masonry piers at high temperatures was also considered of importance.

In planning their investigation, Bauschinger's work was carefully considered by Möller and Lühmann, who concluded that in both the first and second series of tests by the former, the wrought-iron columns were overloaded, resulting in relatively greater bending as the columns deflected under the unsymmetrical heating and giving greater extreme fiber stress than the columns could withstand. It was held that a comparison of cast and wrought iron columns should be made with specimens identical in length, lateral dimensions, and moment of inertia.

The specimens tested by Möller and Lühmann were 3.28 feet (1 m), 6.56 feet (2 m), or 13.12 feet (4 m), in length. The shortest length was chosen to give approximately the compressive strength of the material, the intermediate length was in about the

[4] Verhandlung des Vereines zu Beforderung des Gewerbfleisses, 66, p. 573 and 701; 1887.

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same ratio to the lateral dimensions as of columns in actual use, and the longest was taken as representing unusually large values of that ratio.

The cast and wrought iron specimens, all approximately 9.8 square inches in cross-sectional area, comprised solid and hollow cylindrical forms, the former 3½ inches in diameter, the latter about 6 inches in outer diameter with wall thickness slightly less than ⅝ of an inch; a hollow, cast-iron, ornamental, fluted column of approximately the same diameter and area as the other hollow sections; and a riveted form of rectangular section (6 by 7 inches) built up of four angles united on two sides by plates and on the other sides by lattice bars, the component parts being rigidly bolted and riveted together. The cast iron columns had fairly uniform wall thickness.

As adding somewhat to the resistance to fire and water, one of the wrought-iron and two of the cast-iron columns were filled with cement, a 2-inch gas pipe being placed concentrically in one of the latter to maintain alignment of the pieces when the column broke. Quite full protection was given one each of the hollow cast and wrought iron specimens by a covering of 1:3 cement mortar about 2½ inches thick, interlaced with wire. Also, one of the riveted angle columns was protected by a circular wood mantle about 1¼ inches thick, covered with sheet iron.

There were further included oak and fir columns 6 inches square and columns of brick 9 inches square set in cement mortar.

Lengths other than 6.56 feet were used only in the unprotected cylindrical cast and wrought iron specimens. Report was made on about 40 tests, of which approximately two-thrids were fire and water tests, the rest being load tests at normal temperature.

The testing apparatus was very similar to that used by Bauschinger, the columns being loaded in a horizontal hydraulic press and heat applied by a coke fire in a U-shaped trough placed beneath the column. Temperatures of 330, 440, and 600° C were measured by metals and alloys placed in contact with the outside of the columns. Vertical deflections were measured at the mid-point of the length, using a lever with one of its ends resting on the column.

The loads were applied with the column axis uniformly 0.39 inch (1 cm) below that of the machine, the authors holding that central loading of columns does not occur in buildings except by chance, and that the above eccentricity is about what may be expected in practice.

In general the tests were conducted with the column ends unrestrained, the machine being fitted with spherical bearing blocks, but to determine the difference a few columns were loaded between fixed parallel bearing plates. Some of the columns were loaded at ordinary temperatures, and the safe load, as judged apparently by the limit of permissible deflection, and the maximum load were determined. The same quantities were determined for the high temperature condition, in general after 330 and 600° C had been attained on the upper and lower sides of the column, respectively. The load determinations at high temperature were generally made when the maximum deflection had been induced, which occurred at the time of water application.

On application of maximum load while hot, the short hollow columns (1 m long) deflected upward due to yielding of the metal at higher temperature on the lower side of the column. The solid columns of this length, in common with all of the longer columns, deflected downward due to the uneven heating, the deflections increasing on application of water and load. The following tabulation gives a summary of the results obtained with unprotected hollow and solid columns of cast iron and wrought iron:

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Form and material	Effective length	Slenderness, ratio $\frac{l}{r}$	Fire and water test		Test at normal temperature	
			Maximum load	Deflection	Maximum load	Deflection
	Feet		Lbs./in.²	In.	Lbs./in.²	In.
Hollow cast iron	4.2	27	16 500	0.98	43 700	1.62
Hollow wrought iron	4.2	27	13 500	1.97	.....	.....
Hollow cast iron	7.5	47	23 000	1.77	37 200	.39
Hollow wrought iron	7.5	47	10 800	1.97	23 000	.78
Solid cast iron	4.2	57	9220	1.17	39 300	.33
Solid wrought iron	4.2	60	9100	.28	22 100	.63
Solid cast iron	7.5	101	6640	1.93	10 900	.98
Solid wrought iron	7.5	107	5000	1.97	13 500	.18
Hollow cast iron	14.0	89	2650	4.75	15 380	1.77
Solid cast iron	14.0	190	1890	6.80	.....	.....
Solid wrought iron	14.0	201	1620	4.56	.....	.....

The decrease in strength with length was marked, particularly for the columns loaded after fire exposure and water application. This was due to the larger lateral deflections caused by unsymmetrical heating, cooling, and loading induced in the longer columns, test conditions that can hardly be said to duplicate typical fire conditions even for unprotected exterior wall columns. In the shorter lengths, the cast iron sustained higher average unit loads than the wrought iron, the difference becoming smaller with increase of length or slenderness ratio. The load reported as safely carried after fire and water application varied from a little over one-half of the maximum load finally sustained to nearly the full value of the latter. Failure was due in all cases to bending produced mainly by uneven temperature distribution over the column section. The tests developed no cracking of the hot cast iron due to water application except in the case of the longest column, in which case the crack could have been incidental to failure as caused by the large deflection.

The columns tested between fixed parallel plates withstood higher loads than those tested with spherical end blocks, although the tests were too few in number to afford definite comparison.

The filled columns sustained slightly higher loads under the same test conditions than the corresponding unfilled columns. The metal of the columns protected by mortar was only moderately warm after the column had been in the fire about 1½ hours, and that of the column protected by wood was 120° C after the covering had been removed subsequent to a 63-minute fire exposure. The strength of the brick columns was reduced about 50 per cent after 40 to 45 minutes' fire exposure. The oak column failed due to reduction of area and strength of the wood after 8 minutes under load of 1240 pounds per square inch, as computed on the original area, and the fir column after 18 minutes in the fire under load of 1110 pounds per square inch.

3. HAMBURG TESTS[1]

Two series of fire tests on loaded building columns were undertaken at Hamburg, Germany, in the period 1893 to 1895, by representatives appointed by the Hamburg

[1] Vergleichende Versuche uber die Feuersicherheit von Speicher-stutzen. Commissions--bericht Hamburg, 1895.
Vergleichende Ubersicht uber die Feuersicherheit gusseiserner Speicher-stutzen. Hamburg, 1897.
Zeitschrift d. Verein Deutch Ing., 40, p. 159; 41, p. 1007; 42, p. 183.
Stahl und Eisen, 18, p. 696.

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senate from the building commission, the fire department, the warehouse association, and the insurance interests.

(*a*) FIRST SERIES.—In the first series tests were made on wrought-iron columns built up of four angles united by latticed bars and plates to form rectangular sections. A few columns were tested unprotected, two were filled with cinder concrete, and nine were protected by coverings from ¾ to 2 inches in thickness, consisting of concrete (Monier) blocks, gypsum and magnesite blocks or boards, pressed cork, and asbestos cement. Some of the coverings were protected by a sheet-metal mantle. In the series were also included three wooden columns 11.8 inch square, one of which was covered by sheet metal.

The question of the relative fire resistance of the several column materials was still of interest, and, although fire-protective coverings had come to be employed, their use had not gained such headway as in America, where hollow tile, at that date not manufactured in Germany, was the almost universal covering material. The importance of simulating in the tests actual loading and fire conditions was recognized.

The columns were loaded by a hydraulic ram in upright position within a framework in which were inserted two platforms 11½ feet apart, to which the test column was attached. Heat was applied over a length of about 4 feet in the middle portion of the column by a gas-fired oven built in halves which swung together about the column. Temperatures were measured by Seger cones, fusible alloys, and a thermoelectric pyrometer, and lateral deflections with rods passed through holes in the furnace walls. Working load was applied, centrally or eccentrically, and heating continued until the column was unable to sustain it, when water was applied over the heated length. The furnace temperature rise varied considerably between different tests and was at times quite rapid, 1200° C being sometimes attained in 2 hours and 1400° C in 4 hours.

The unprotected wrought-iron columns of the first series failed after 17 to 59 minutes' fire exposure, depending on the rate of temperature rise, the metal temperature at failure being given at about 600° C, and the load sustained 14 200 pounds per square inch (1000 kg per sq. cm). The filled columns proved to be only slightly more resistive than the protected and unfilled columns. The protected wrought-iron columns failed after 1¾ to 4 hours, with maximum furnace temperature of 1200 to 1300° C, the load applied being apparently the same as for the unprotected columns. The temperatures inside of the column were not known, except that they were over 412° C.

The application of water resulted in injury to the concrete coverings, and distruction of the other coverings, except where covered with sheet metal.

The wooden columns failed after a furnace exposure averaging a little over 1 hour for the three tests, the reduction in cross section from combustion of the wood being about 40 per cent.

(*b*) SECOND SERIES.—Tests were made on about 24 columns of cast iron of 10.8 inch outside diameter, 22 of which had wall thicknesses of 1.18 inch, and two 0.47 inch, the columns being cast in vertical position. Of the total number 17 were protected by coverings 1½ to 2 inches (4 to 5 cm) thick consisting of the materials used in the first series, to which were added coverings of tufa stone and asbestos-

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kieselguhr. A number of the coverings were encased in sheet metal. The effect of an air space between the covering and column and of free convection within the column were studied, as, also, the difference between permanent as distinguished from removable coverings, some members of the commission holding that coverings should be removable to permit examination of the column from time to time on account of danger from rust.

The method of testing was substantially the same as in the first series. The unprotected cast-iron columns failed in from 35 to 59 minutes, depending on the intensity of the fire, under load of 7100 pounds per square inch (500 kg per sq cm), and temperature of the column reported as 800° C. Increasing the column load to 10 700 pounds per square inch (750 kg per sq cm) caused failure to occur with column temperature of 700° C. The thin-walled column gave a little lower resistance than those of greater thickness. The application of water to unprotected columns that had failed in the fire test and suffered large deformation generally caused cracking.

The protected columns failed after 3 to 5½ hours, with maximum furnace temperatures from 1200 to 1500° C, except the one covered with 2-inch thickness of asbestos-kielselguhr, which did not fail after a fire exposure of 7 hours. The air space between the column and the covering increased the fire resistance a little and the provision for free convection within the column delayed failure by nearly one hour. The action of water on the coverings was the same as in the first series.

(*c*) GENERAL RESULTS.—The Hamburg tests were the first to determine the strength of wrought iron and cast iron, applied as columns, under central load and symmetrical heating. The column temperatures reported, regarded as those of the metal at failure, were without doubt higher than the actual temperatures, due to the methods of measurement employed. As column tests they are open to objection in that only about one-third of the column length was heated. The tests on protected columns while giving little information on the effectiveness of covering materials now in use, proved conclusively the great gain in fire resistance attainable by protecting the metal with materials of low heat conductivity.

With regard to water application on cast-iron columns at high temperature, the results are not conclusive, since in almost all tests the columns had sustained large deformations by failure in the fire test before water was applied.

4. FIRE TEST OF COLUMN AT VIENNA[6]

In 1893 the Building Department of Vienna conduced a fire and water test on a single wrought-iron column 11½ feet long, built up of two 5½-inch channels connected by lattice bars. The column was protected by brickwork 5½ inches thick, laid in fire clay mortar, the interior of the column being unfilled.

The furnace was of brick 8 by 12 feet, and with height equal to the column length. Wood piled 3 to 4 feet high was used for fuel, and the temperatures of furnace and column were indicated by alloys. Load was applied to the column section, using a long lever. After a fire test of 2½ hours duration, wherein furnace temperatures evidently much in excess of 400° C were attained, a hose stream was applied to one side of the column.

The only apparent damage caused by the combined test was considerable spalling of the bricks at the corners, cracking of bricks in the upper half of the covering, and washing away of some of the mortar. The maximum temperature indicated inside of the column was 65° C.

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[6] Engineering News, 32, p. 184; Sept. 6, 1894. Translation of article in Zeitschrift des Oesterreicheschen Ingen. u. Arch. Verein.

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5. NEW YORK TESTS[1]

Fire tests of unprotected columns, two of structural steel and two of cast iron, and a fire and water test of one unprotected cast-iron column, were made in 1896 under the direction of a committee appointed by the Tariff Association of New York, the Architectural League of New York, and the American Society of Mechanical Engineers.

The steel columns were 14 feet long, one being a box section built up of two 10-inch channels and two 12 by ¼ inch plates, and the other a Z-bar section consisting of four 4 by $\frac{7}{16}$ inch Z-bars riveted to a plate 6½ inches wide, other details of design being in accord with accepted standards of practice. The cast-iron columns were 13 feet long, of 8-inch outside diameter and 1-inch wall thickness, and were cast in horizontal position with dry-sand core, the ends being flanged and end bearings machined.

The columns were tested in vertical position within a furnace chamber about 12 feet square and 14 feet high, the fuel used being producer gas admitted by burners extending through the floor of the furnace. Air was supplied through openings in the floor near the gas inlets. Arrangements were made for intensifying the fire when necessary by injecting a naphtha spray into the gas main supplying the burners. Load was applied to the column within a restraining frame of structural steel by a hydraulic ram placed beneath the column, filler blocks of cast iron transmitting the load to the column.

In the fire tests it was intended to subject the columns to working loads, and for one test each of steel and cast-iron columns to use a slow and a rapid furnace temperature rise. Furnace temperatures were measured with an Uhling-Steinbart transpiration pyrometer. Temperatures on the metal of the columns were not measured in any of the tests.

In the test of the plate and channel column, using a slow temperature rise, a furnace temperature of about 650° C was attained in one hour, which temperature maintained for about 20 minutes caused the steel to show red color and fail under a load of 6400 pounds per square inch, trouble with the loading equipment preventing the application of the full working load.

The Z bar column, tested under full working load of 12 000 pounds per square inch and rapid furnace temperature rise, failed 25 minutes after the beginning of the test. A maximum furnace temperature of about 750° C was indicated at 14 minutes, which fell off to 600° C at failure.

The cast-iron columns were tested under working load of 7700 pounds per square inch. In the test with slow temperature rise the column began to show color after 65 minutes' fire exposure, the furnace temperature being near 600° C. After another 18 minutes, during which the furnace temperature averaged 640° C, the gas was shut off and the furnace door opened for 9 minutes, which disclosed the test column to be decidedly red and bent. The door was closed and the test continued at lower furnace temperature for 28 minutes, the column sustaining its load, although greatly bent. No cracks developed. The permanent lateral deflection was 3½ inches.

The cast-iron column tested with rapid temperature rise had deflected a visible amount at 35 minutes and began to show color at 39 minutes, the furnace temperature at this time being 730° C. It failed at 43 minutes with complete fracture across the section near the middle.

In the fire and water test, water was applied at four successive times following fire exposures of increasing intensity. Before the third application the furnace temperature was 580° C, and the column showed color. Immediately before the last water

[1] Trans. Am. Soc. of Mech. Eng., 18, p. 24, 1896–1897.

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application a furnace temperature of 700° C was indicated, and the column was red and bent. No cracks developed. The permanent lateral deflection was 3¼ inches.

The tests made apparent that unprotected metal columns will fail in fires of moderate intensity after a comparatively short exposure, the steel columns being less resistive than those of cast iron under the respective unit loads for which the two types are generally designed. Injury to cast-iron columns due to water application while hot is shown to be improbable with columns of the given proportions. The tests give little information on the temperature in the metal at failure or on definite time resistance under comparable fire conditions.

6. WAITE'S TESTS

In 1903 the Guy B. Waite Co., of New York, conducted some fire and water tests on floors and partitions, under the supervision of the Manhattan Bureau of Buildings, constructing to that end a reinforced-concrete test house, in which were placed, for the purpose of studying incidentally the behavior of column coverings, a reinforced-concrete column made with stone aggregate, proportion of mixture, 1 : 2 : 4, and two cast-iron columns, one of which was covered with 1½ inches of poured cinder concrete of proportion 1 : 5, Portland cement, and hard-coal cinders, the other with a gypsum and cinder composition.

The reinforced-concrete column was considerably pitted by water from a 1½-inch nozzle at 60 pounds' pressure after it had been subjected for 4 hours to a test fire with average temperature of about 930° C. The cinder-concrete covering was in excellent condition after it had gone through the same test, and, in addition, three tests of 1-hour duration with water application at the end of each. The gypsum covering appeared in good condition at the end of a single fire test, but on water application it was partially washed away.

7. TESTS BY McFARLAND AND JOHNSON[8]

Tests to determine the effect of fire and water on the strength of reinforced-concrete columns were made in 1906 by H. B. McFarland and E. V. Johnson at the Chicago laboratory of the National Fireproofing Co.

The columns, three in number, about 10½ inches square and 12 feet long, were made of 1 : 2 : 4 limestone concrete and reinforced with ⅜-inch rods placed near the corners. Two columns were tested in compression at normal temperature at age of 2 months and 3½ months, respectively, and sections 5 to 6 feet long cut from them outside of the region of failure for use in the fire test. One of these specimens was covered with 3-inch solid porous clay tile and the other was tested unprotected, and after the fire test subjected to water application. The third column was cut in two, one part for use in the fire test and the other for a comparable compression test at normal temperature. The age of all columns was 23 months at the time of the fire test.

The three specimens were placed on end in a wood-fired furnace and subjected under no load to a 3-hour fire test. Furnace temperatures, as indicated by a Bristol thermoelectric pyrometer, ranged from 800 to 1000° C for the greater portion of the period. On the day following the fire test they were tested in compression.

The section protected by clay tile was little affected by the test and developed compressive strength of 3127 pounds per square inch. An 18-inch long specimen cut from the same column but not subjected to fire test developed 3558 pounds per square inch. The specimen to which water was applied after the fire test failed in the compression test at 674 pounds per square inch. The column from which it was cut had

[8] Engineering News, 56, p. 316; Sept. 20, 1906.

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developed 2116 pounds per square inch at age of 3½ months, the fire and water treatment having apparently caused a decided decrease in strength. A similar effect was indicated in the case of the third specimen, its strength being 711 pounds per square inch, against 2565 pounds per square inch for the section of the same column that was not exposed to fire.

The large reduction in strength sustained by the unprotected columns can be ascribed, in part, to their small size, larger columns being subject to smaller percentage reduction in strength due to surface damage from fire exposure.

WASHINGTON, June 4, 1920.

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APPENDIX F

CENTIGRADE AND FAHRENHEIT CONVERSION TABLE

C.°	0	10	20	30	40	50	60	70	80	90
	F.°	F.°	F.°	F.°	F.°	F.°	F.°	F.°	F.°	F.°
0	32	50	68	86	104	122	140	158	176	194
100	212	230	248	266	284	302	320	338	356	374
200	392	410	428	446	464	482	500	518	536	554
300	572	590	608	626	644	662	680	698	716	734
400	752	770	788	806	824	842	860	878	896	914
500	932	950	968	986	1004	1022	1040	1058	1076	1094
600	1112	1130	1148	1166	1184	1202	1220	1238	1256	1274
700	1292	1310	1328	1346	1364	1382	1400	1418	1436	1454
800	1472	1490	1508	1526	1544	1562	1580	1598	1616	1634
900	1652	1670	1688	1706	1724	1742	1760	1778	1796	1814
1000	1832	1850	1868	1886	1904	1922	1940	1958	1976	1994
1100	2012	2030	2048	2066	2084	2102	2120	2138	2156	2174
1200	2192	2210	2228	2246	2264	2282	2300	2318	2336	2354
1300	2372	2390	2408	2426	2444	2462	2480	2498	2516	2534
1400	2552	2570	2588	2606	2624	2642	2660	2678	2696	2714
1500	2732	2750	2768	2786	2804	2822	2840	2858	2876	2894
1600	2912	2930	2948	2966	2984	3002	3020	3038	3056	3074
1700	3092	3110	3128	3146	3164	3182	3200	3218	3236	3254
1800	3272	3290	3308	3326	3344	3362	3380	3398	3416	3434
1900	3452	3470	3488	3506	3524	3542	3560	3578	3596	3614
2000	3632	3650	3668	3686	3704	3722	3740	3758	3776	3794

C.°	F.°
1	1.8
2	3.6
3	5.4
4	7.2
5	9.0
6	10.8
7	12.6
8	14.4
9	16.2
10	18.0

F.°	C.°
1	.56
2	1.11
3	1.67
4	2.22
5	2.78
6	3.33
7	3.89
8	4.44
9	5.00
10	5.56
11	6.11
12	6.67
13	7.22
14	7.78
15	8.33
16	8.89
17	9.44
18	10.00

Examples:

515° C.= 950° F.+9° F.= 959° F.
2039° F.=1110° C.+5° C.=1115° C.

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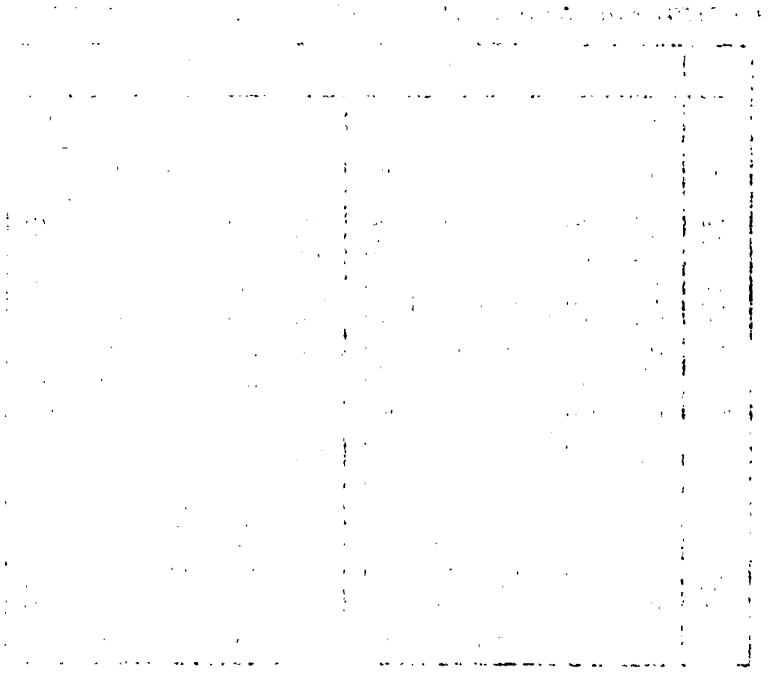

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

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No. 185

EXPERIMENTS ON COPPER CRUSHER CYLINDERS

BY

ALEXANDER I. KRYNITSKY, Associate Physicist
Bureau of Standards

FEBRUARY 1, 1921

PRICE, 10 CENTS
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EXPERIMENTS ON COPPER CRUSHER CYLINDERS

By Alexander I. Krynitsky

ABSTRACT

The experiments consisted of compression tests on a standard Riehlé testing machine. The results of these compressions carried out on annealed copper cylinders, one set having a mean length of 0.4000 inch and the others 0.5000 inch, with a mean diameter of 0.2260 inch in the former and 0.2500 and 0.2250 inch in the latter ones, suggest that the length of the cylinders decreased considerably under repeated application of the same and, within certain limits, of smaller loads.

In the case where the load is applied in such a manner as to produce the maximum stress for only an instant, the duration of an application of load has but little effect on the decrease in length of the cylinders, but the application of the same load by holding the beam balanced through an additional application of this load causes about twice the decrease in length as compared with that obtained by a single application of load. When two successive loads of considerable amount differing by increments of about 2000 pounds per square inch are applied, the second being greater than the first, the change in length due to the last load is greater than that obtained where the pressure is applied on previously uncompressed cylinders, and this difference decreases as the difference between the two loads successively applied increases, so that when the difference between the loads reaches a certain value the change of length due to the last load applied is practically independent of the previous load.

It appears probable that aging at temperatures within the range 0–100° C softens the compressed copper somewhat.

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I. INTRODUCTION

1. TWO METHODS FOR MEASUREMENT OF PRESSURE OF GASES IN TESTING OF POWDER AND AMMUNITION

Among many methods and apparatus [1] proposed for the measurement of pressure developed by the products of decomposition of powder in the testing of powder and ammunition there are two methods which up to the present time are commonly employed in practice; that is (1) the so-called dynamic method and (2) the method of measuring pressure by means of metallic cylinders, usually copper or lead cylinders, known as crusher cylinders.

The dynamic method consists of determining the velocities of the gun in recoil or of the shot at different points of the bore. The difference of the velocities divided by the corresponding differences of the time equal the acceleration. The pressure required to produce the observed acceleration in a body whose mass is that of the gun or of the projectile is obtained by multiplying the acceleration by the mass.

The crusher-gage method was proposed in 1868 by Noble, and up to the present time is regarded as the most convenient known method for obtaining an idea of the relative values of pressures in guns.

There is a metal cylinder set on an anvil. On firing, the pressure of the gases shortens the cylinder longitudinally. The amount of this compression, with a previously prepared table, so-called tarage table, serves to determine the maximum pressure of the products of decomposition. For the testing of small arms a special barrel is used. For example, in Figs. 1 and 2, a pressure-gage outfit

[1] It may be mentioned here that two types of registering apparatus proposed very recently appear very promising, and they possibly will find common use in the near future. One of these was proposed by Prof. A. G. Webster, of Clark University, and the other is designed to coordinate time and pressure within a large cannon, upon which Dr. H. L. Curtis and associates, of the electrical division of the Bureau of Standards, are working.

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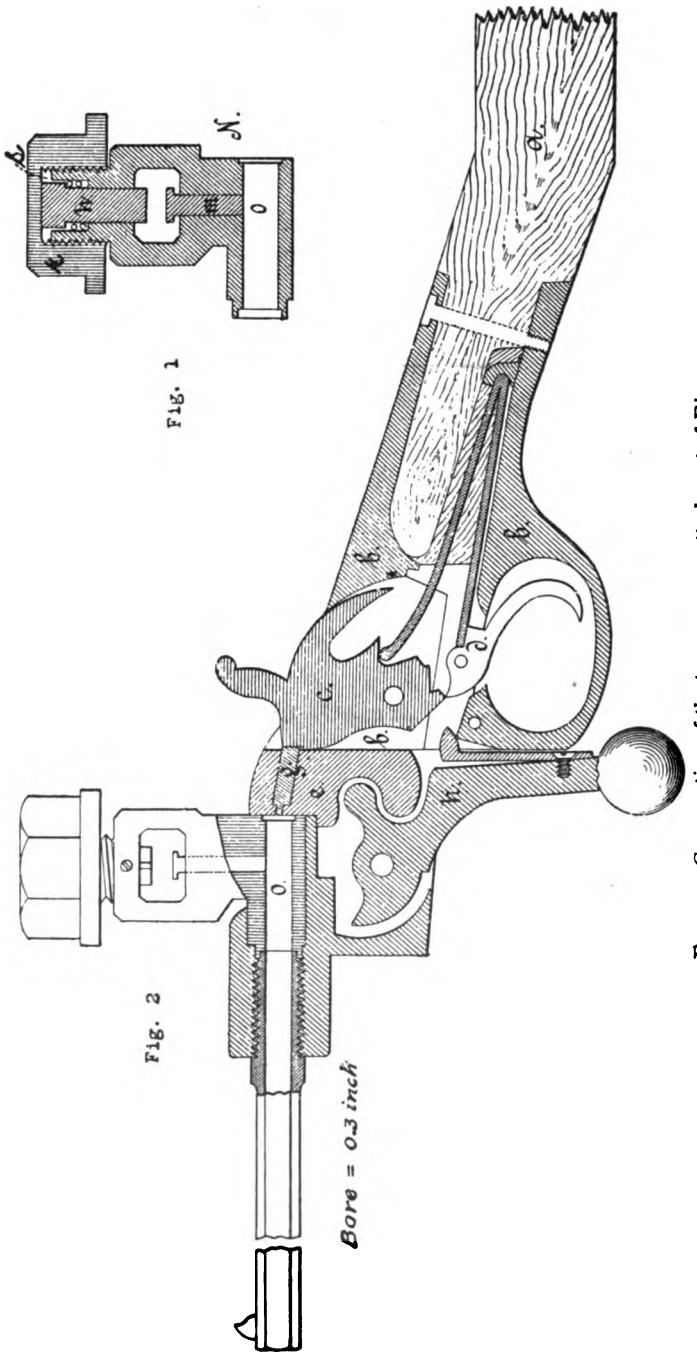

FIG. 1.—*Cross section of the pressure-gage attachment of Fig. 2*
FIG. 2.—*A pressure-gage outfit for Russian 0.3-inch revolver*

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for Russian 0.3-inch revolver is shown: O = a special barrel; h = an unmovable piston fixed rigidly in its position by means of nut r and spring s; m = a movable piston. Between pistons h and m crusher cylinders are placed. By turning the nut r the crusher cylinder may be tightened between these two pistons.

2. COPPER CRUSHER CYLINDERS

In the present paper only copper crusher cylinders will be considered. Dimensions of copper cylinders vary in different countries and for pressure testing of different grades of ammunition and powder.

For example, some dimensions of copper crushers are as follows:

Dimensions	A	B	C	D
	mm	mm	Inch	Inch
Diameter	3	8	0.2260	0.2520
Height	4.9	13	.4000	.5000

The copper must be exceptionally pure, and the whole process of manufacture of copper crushers should be standardized with extreme care.

3. PREPARING DATA FOR TARAGE TABLE

In preparing these data the copper cylinders are slowly compressed by applying the loads and measuring the amount of compression at each load. The load for each compression remains active for about 15 seconds. Ch. B. Wheeler,[2] from his experiments, found it might safely be assumed that at the end of 15 seconds the copper is practically in equilibrium with its load and that any further application of load produces no effect.

Influence of the friction of the testing machine is of great importance in the construction of tarage tables. As early as 1891 this was demonstrated by Vieille, who showed that neglect of this factor was responsible for the erroneous French crusher tables.

This point, states F. W. Jones,[3] must be insisted upon because experiments in some quarters assume friction is of no account, a conclusion to which they are driven because no provision has been made in the design of their testing machine for applying the load with friction acting alternately back and forth.

[2] Report of Experiments with an 8-inch B. L. Rifle Mounted on a Free Recoil Carriage, by Lieut. Ch. B. Wheeler, Ordnance Construction, Notes 69–85, pp. 46–56; Jan., 1896.

[3] A Standard Pressure Table for Copper Crushers, by F. W. Jones, Arms and Explosives, June 1, 1918.

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4. COMPARISON OF PRESSURES INDICATED BY DYNAMIC AND CRUSHER-GAGE METHODS

The dynamic method shows a pressure which is less than the true pressure expected in the gun, as it takes no account of the pressure required to overcome the friction of the projectile in the bore and to rotate the projectile.

Comparing now the dynamic method and the crusher-gage method, we find that the latter gives pressures which are less than those shown by the dynamic method.

According to B. W. Dunn,[4] under ordinary service conditions (since the time required by powder pressure to reach a maximum is less than 0.003 second or about 0.001 second) the probable errors with the crusher-gage method will be greater than 5000 pounds per square inch.

The quicker the powder the greater is the excess of actual over indicated pressures and, for this reason, the error of the crusher gage is greater for small arms than for cannon.

5. NECESSITY OF PRELIMINARY PRECOMPRESSION

Though the crusher-gage method is very old, there remains up to the present time a problem of great importance—the necessity of precompression. It has been stated by many experimenters that copper cylinders should be precompressed at a certain pressure before using for test. Capt. Blunt,[5] F. W. Jones,[6] Marshall,[7] and others state that copper should be compressed beforehand with a static pressure only slightly less than that which it is to undergo in the actual test.

Col. W. H. Tschappat[8] states that when the pressure in the gun is high and quickly developed, it is at first very much greater than the resistance offered by the copper cylinder. The excess of pressure over resistance in this case accelerates the mass of the piston and develops considerable velocity in it. The energy of the piston due to this velocity is finally absorbed by further compression of the copper; but in the early stages of compresssion the mass of the piston prevents as great a compression as would be

[4] Report of Development of a Photoretardograph and Its Application to the Dynamic Measurement of Resistance to Compression Offered by the Copper Cylinders Used in Crusher Gages, by Lieut. B. W. Dunn, Ordnance Construction Notes, pp. 69–85; Jan. 13, 1896.

[5] Report of Capt. Blunt, Report of Chief of Ordnance, pp. 929–943; 1892.

[6] F. W. Jones, Lectures to Young Gunmakers, Arms and Explosives, p. 58, April, 1901, and p. 139, March, 1912.

[7] Marshall, Explosives, p. 363.

[8] Lieut. Col. W. H. Tschappat, Ordnance and Gunnery, pp. 101–103; 1917.

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obtained if the pressure acted directly upon the copper cylinder instead of through the piston.

It is therefore desirable to reduce the energy of the piston as much as possible, and this may be done in two ways: By reducing its weight and by limiting its travel and accompanying velocity. As far back as 1883 Sarrau and Vieille [9] [10] showed that deformation of copper cylinders depends upon the weight (mass) of the piston through which the pressure is communicated. Usually the piston is made as light as possible.

To limit the travel all copper cylinders are precompressed by a load, generally about 3000 pounds per square inch less than that expected in the gun. According to many statements, initially compressed cylinders always record more uniform results than the uncompressed cylinders do.

Many investigators are of the opinion that the influence of many factors which have an important bearing on the results of pressure testing are minimized by the use of initially compressed cylinders, and they believe also that it is to their advantage to check each cylinder by an initial compression. On the other hand, there is rather good evidence that very uniform results may be obtained with uncompressed cylinders when they are made extremely uniform.

Thus, as was said above, the necessity of precompression up to the present time is rather an open question. In order to obtain some more data for the solution of this problem, experiments were started by the Bureau of Standards in connection with the standardization of the crusher-gage method for pressure testing of small arms, ammunition, and powder.[11] So far, the experiments have been confined to static tests; these are described in this paper. The experiments were carried out in 1919; they consisted of a series of compressions of the cylinders on a standard 10 000-pound Riehlé Brothers testing machine, and some microscopic examinations of their longitudinal and cross sections. The results are given in the tables and diagrams, so that few explanatory notes are required.

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[9] Sarrau et Vieille, Etude sur l'Emploi des Manomètres pour la Mesure des Pressions Développées par le Substances Explosives.

[10] The Interior Ballistic, by Col. Pashkevitch, translated from Russian by Lieut. Tasker H. Bliss.

[11] The standardization of the crusher-gage method was started by Dr. P. D. Merica under the supervision of Dr. G. K. Burgess.

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II. COMPRESSION TESTS

1. METHODS OF COMPRESSION

The rate of compression was always about 0.07 inch per minute. Duration of load after balance was obtained in most cases was about 2½ seconds, during which time the beam was not falling down.

Two methods of applying the load were used:

(1) There was single application of load. As soon as balance was obtained, the load was removed; or in some cases the load was removed after a certain period—5 or 25 seconds.

(2) After the balance was obtained, which was always followed by drop of the beam about 3 seconds after original balance, the balance of beam was maintained by reapplying the same load.

2. MEASUREMENT

The measurements were then made with a 1/10 000-inch micrometer. Measurements of length were taken with the cylinders placed in the micrometer centrally, each length being measured several times and only the minimum length recorded. Diameters were measured several times and the average recorded. After compression the cylinders have a barrel shape and in some cases two diameters of each cylinder were measured; that is, diameter of base; also diameter of cross section midway between the bases.

The diameter of the cross section midway between the bases is called "maximum" in the tables, and, as was stated, it is an average value of several measurements. The diameter of the base is called "minimum" and is also an average of several measurements. The minimum diameter, because of the difficulty of obtaining the exact measurement, is only an approximate value.

3. PLAN OF DESIGNATION

There are two lots of copper crusher cylinders both annealed at 1200° F at the same plant, but at different times. These two lots are here called Series No. 1 and Series No. 2.

There are also two other sets annealed at another factory at 1000° F and 1200° F, respectively, and called Series No. 3. The dimensions are given in inches, time in seconds, weight in pounds. Certain figures omitted were disregarded in drawing up the average as doubtful for various reasons. The following symbols will be used:

L = initial length of cylinders.

D = initial diameter of cylinders.

A = initial cross-sectional area.

L_1 = length of cylinders after the first compression.

L_2 = length of cylinders after the second compression, and so on.

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D_1 = diameter of cylinders after the first compression.
D_2 = diameter of cylinders after the second compression, and so on.
$L-L_1$ = change in length of cylinders after first compression.
$L-L_2$ = change in length of cylinders after the second compression, and so on.

4. RESULTS OF COMPRESSION TESTS

(a) EXPERIMENTS ON REPEATED APPLICATION OF SAME AND OF SMALLER LOADS

TABLE 1.—Experiments on Repeated Application of Same and of Smaller Loads

L_1 = length of cylinders after applying pressure of 30 000 lbs./in.² for about 30 seconds.
L_2 = length of cylinders after applying pressure of 40 000 lbs./in.² for about 30 seconds.
L_3 = length of cylinders after applying pressure of (Note a) and (Note b) lbs./in.² for about 30 seconds.
L_4 = length of cylinders after applying pressure of 38 000 lbs./in.² for about 2½ seconds.
L_5 = length of cylinders after applying pressure of 36 000 lbs./in.² for about 2½ seconds.
L_6 = length of cylinders after applying pressure of 34 000 lbs./in.² for about 2½ seconds.
L_7 = length of cylinders after applying pressure of 30 000 lbs./in.² for about 2½ seconds.
L_8 = length of cylinders after applying pressure of 25 000 lbs./in.² for about 2½ seconds.

No.	L	D	A	Load equivalent to 30 000 lbs./in.²	$L-L_1$	D_1 maximum	Load equivalent to 40 000 lbs./in.²	$L-L_2$	D_2 maximum	Load equivalent to—(See [a] or [b])	$L-L_3$	Load equivalent to 38 000 lbs./in.²
	Inch	Inch	Inch²	Lbs.	Inch	Inch	Lbs.	Inch	Inch	Lbs.	Inch	Lbs.
1	0.5004	0.2514	0.0495	1485	0.0494	0.2672	1980	0.0814	0.2780	[a] 1980	.....	1881
2	.5009	.2516	.0497	1491	.0490	.2675	1988	.0786	.2778	[a] 1988	.....	1888
3	.5003	.2516	.0497	1491	.0484	.2675	1988	.0786	.2768	[a] 1988	.....	1888
4	.5006	.2517	.0497	1491	.0485	.2678	1988	.0802	.2781	[a] 1988	.....	1888
5	.5004	.2514	.0495	1485	.0487	.2669	1980	.0786	.2772	[a] 1980	.....	1881
6	.4996	.2516	.0497	1491	.0482	.2673	1988	.0799	.2784	[b] 1888	0.0816	1888
7	.5009	.2517	.0497	1491	.0482	.2677	1988	.0799	.2780	[b] 1888	.0810	1888
8	.5010	.2517	.0497	1491	.0501	.2673	1988	.0794	.2777	[b] 1888	.0823	1888
9	.5013	.2518	.0497	1491	.0480	.2672	1988	.0786	.2774	[b] 1888	.0808	1888
10	.5003	.2517	.0497	1491	.0492	.2673	1988	.0798	.2777	[b] 1888	.0820	1888
Average	.....	.....	.....	.....	.0488	.....	.....	.0795	.....	.....	.0815	.....

No.	$L-L_4$	D_4 maximum	D_4 minimum	Load equivalent to 36 000 lbs./in.²	$L-L_5$	Load equivalent to 34 000 lbs./in.²	$L-L_6$	Load equivalent to 30 000 lbs./in.²	D_7 maximum	D_7 minimum	$L-L_7$	Load equivalent to 25 000 lbs./in.²	$L-L_8$
	Inch	Inch	Inch	Lbs.	Inch	Lbs.	Inch	Lbs.	Inch	Inch	Inch	Lbs.	Inch
1	.....	0.2790	0.2669	1782	.....	1683	.....	1485	0.2795	0.2679	.....	1237	.....
2	.....	.2790	.2669	1789	.....	1690	.....	1491	.2795	.2675	.....	1243	.....
3	.....	.2780	.2660	1789	.....	1690	.....	1491	.2785	.2675	.....	1243	.....
4	.....	.2800	.2645	1789	.....	1690	.....	1491	.2805	.2675	.....	1243	.....
5	.....	.2790	.2660	1782	.....	1683	.....	1485	.2790	.2675	.....	1237	.....
6	0.0837	.2790	.2650	1789	0.0842	1690	0.0861	1491	.2796	.2675	0.0867	1243	0.0867
7	.0842	.2795	.2650	1789	.0851	1690	.0861	1491	.2788	.2675	.0865	1243	.0872
8	.0843	.2790	.2669	1789	.0850	1690	.0859	1491	.2798	.2675	.0863	1243	.0864
9	.0835	.2780	.2650	1789	.0862	1690	.0877	1491	.2800	.2675	.0880	1243	.0880
10	.0835	.2785	.2660	1789	.0848	1690	.0862	1491	.2795	.2675	.0866	1243	.0868
Av.	.0838	.....	.....	.....	.0851	.....	.0865	.....	.....	.....	.0868	.....	.0870

[a] Load in pounds equivalent to 40 000 lbs./in.² [b] Load in pounds equivalent to 38 000 lbs./in.²

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NOTES ON TABLE 1.—Experiments were made upon copper crusher cylinders of Series I. The first and second compressions were made January 31, 1919. The third compression was made February 13, 1919. All three compressions were made under condition (2), stated above, keeping beam balanced for about 30 seconds after balance was obtained. The rest of the compressions

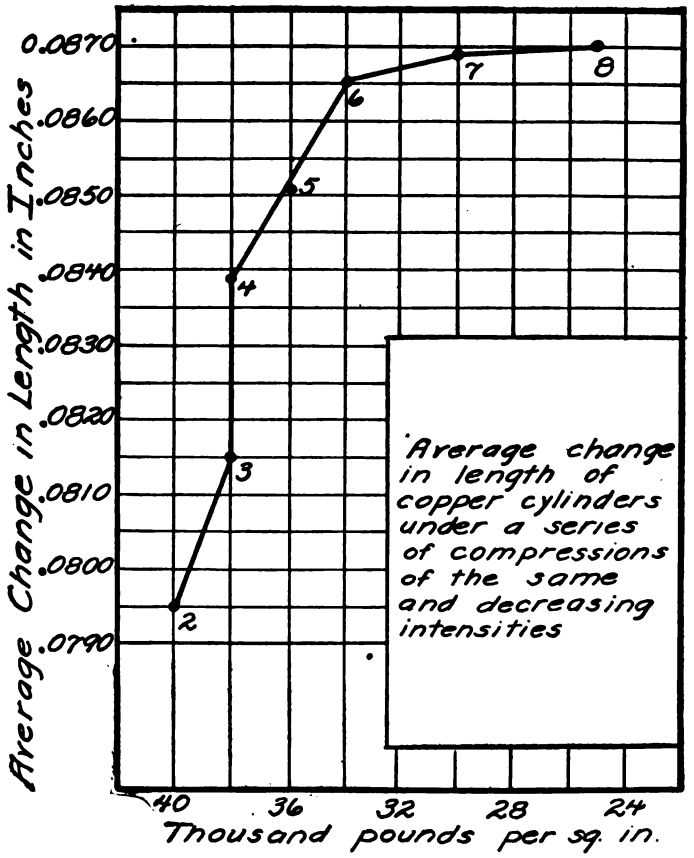

FIG. 3.—*Change of length of copper cylinders upon compression*

(compressions 4–8) were made May 17–19, 1919, under condition (1), with single application of load for about 2½ seconds.

As is shown in Fig. 3, the length of the copper cylinders decreases considerably under repeated application of the same and even smaller loads; this is in spite of the fact that the first three compressions were made keeping the beam in a balanced position for about 30 seconds.

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TABLE 2.—Experiments on Repeated Application of Decreasing Loads

L_1=length of cylinders after applying pressure of 30 000 lbs./in.² for about 30 seconds.
L_2=length of cylinders after applying pressure of 28 000 lbs./in.² for about 2½ seconds.
L_3=length of cylinders after applying pressure of 25 000 lbs./in.² for about 2½ seconds.

No.	L	D	A	Load equivalent to 30 000 lbs./in.²	$L-L_1$	D_1 maximum	Load equivalent to 28 000 lbs./in.²	$L-L_2$	Load equivalent to 25 000 lbs./in.²	L_3	$L-L_3$
	Inch	Inch	Inch²	Pounds	Inch	Inch	Pounds	Inch	Pounds	Inch	Inch
1.....	0.5010	0.2517	0.0497	1491	0.0486	0.2674	1391	0.0491	1243	0.4515	0.0495
2.....	.5001	.2517	.0497	1491	.0496	.2675	1391	.0505	1243	.4496	.0505
3.....	.4999	.2515	.0496	1488	.0482	.2671	1388	.0494	1240	.4502	.0497
4.....	.5010	.2517	.0497	1491	.0475	.2668	1391	.0496	1243	.4500	.0510
5.....	.5006	.2515	.0496	1488	.0481	.2670	1388	.0507	1240	.4499	.0507
6.....	.5002	.2515	.0496	1488	.0482	.2670	1388	.0505	1240	.4487	.0515
7.....	.4998	.2518	.0497	1491	.0468	.2663	1391	.0483	1243	.4505	.0493
8.....	.5000	.2516	.0497	1491	.0468	.2668	1391	.0484	1243	.4505	.0495
9.....	.5005	.2515	.0496	1488	.0467	.2677	1388	.0484	1240	.4503	.0502
10.....	.5003	.2514	.0495	1485	.0482	.2665	1386	.0501	1237	.4493	.0510
Average	.....	.....	.....	.....	.0478	.....	.....	.0495	.....	.....	.0503

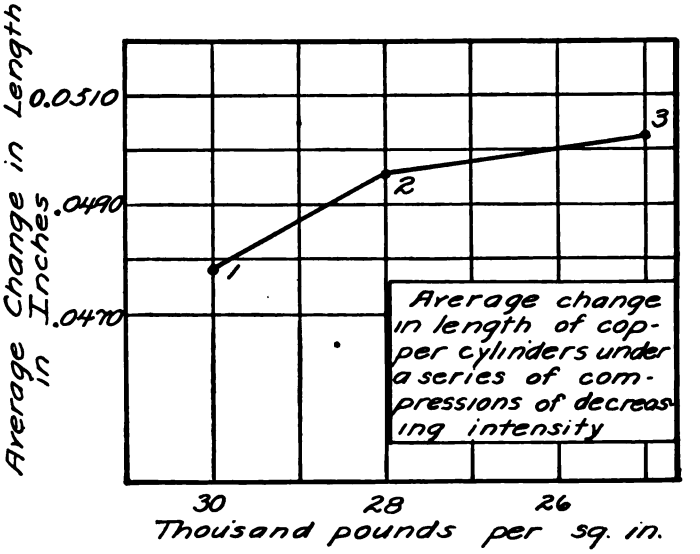

FIG. 4.—*Change of length of copper cylinders upon compression*

NOTES ON TABLE 2.—Experiments were made with copper cylinders of same stock as those of Table No. 1. The first compression was made February 1, 1919, with load of 30 000 pounds per square inch applied under condition (2), keeping the beam balanced for about 30 seconds.

The second and third compressions were made May 20, 1919, with single application of load 28 000 and 25 000 pounds per square inch for about 2½ seconds under condition (1). Fig. 4 indicates that repeated application of decreasing loads gradually decreases length of cylinders (within certain limits).

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TABLE 3.—Experiments on Repeated Application of Same Load

No.	L	D	A	Load equivalent to 40 000 lbs./in.²	D_1 maximum	D_1 minimum	$L-L_1$	$L-L_2$	$L-L_3$
	Inch	Inch	Inch ²	Pounds	Inch	Inch	Inch	Inch	Inch
1	0.4010	0.2265	0.04029	1612	0.2500	0.2350	0.0618	.....	.....
2	.4008	.2260	.04012	1604	.2485	.2350	.0586	0.0616	0.0638
3	.4005	.2256	.03997	1599	.2490	.2350	.0608	.0645	.0658
4	.4004	.2261	.04015	1606	.2468	.2350	.0547	.....	.....
5	.4005	.2262	.04018	1607	.2483	.2350	.0579	.0614	.0640
6	.4002	.2267	.04036	1614	.2498	.2350	.0602	.0630	.0646
7	.4002	.2267	.04036	1614	.2502	.2350	.0603	.0633	.0654
8	.4007	.2262	.04018	1607	.2493	.2350	.0593	.0630	.0657
9	.4004	.2266	.04033	1613	.2494	.2350	.0592	.0624	.0644
10	.4001	.2261	.04015	1606	.2486	.2350	.0587	.0618	.0635
Average	.....	.....	.....	.....	.....	.....	.0592	.0626	.0646

No.	$L-L_4$	D_4 maximum	D_4 minimum	$L-L_5$	D_5 maximum	D_5 minimum	$L-L_6$	$L-L_7$	$L-L_8$
	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch
1	.....	0.2525	0.2386	.....	0.2530	0.2405	.....	.....	.....
2	0.0673	.2514	.2386	0.0687	.2517	.2405	0.0697	0.0709	0.0735
3	.0682	.2514	.2386	.0696	.2522	.2405	.0714	.0736	.0753
4	.....	.2498	.2386	.....	.2505	.2405	.....	.....	.....
5	.0659	.2505	.2386	.0670	.2510	.2405	.0682	.0694	.0703
6	.0670	.2519	.2386	.0702	.2532	.2405	.0708	.0719	.0726
7	.0666	.2520	.2386	.0682	.2534	.2405	.0698	.0721	.0740
8	.0685	.2516	.2386	.0700	.2526	.2405	.0709	.0727	.0746
9	.0675	.2519	.2386	.0693	.2534	.2405	.0717	.0734	.0748
10	.0665	.2508	.2386	.0675	.2518	.2405	.0693	.0710	.0732
Average	.0672	.....	.....	.0688	.....	.....	.0702	.0718	.0735

TABLE 4.—Experiments on Repeated Application of Same Load

No.	L	D	A	Load equivalent to 32 000 lbs./in.²	D_1 maximum	D_1 minimum	$L-L_1$
	Inch	Inch	Inch²	Pounds	Inch	Inch	Inch
1	0.4001	0.2260	0.04012	1284	0.2408	0.2295	0.0410
2	.3998	.2260	.04012	1284	.2408	.2295	.0389
3	.3999	.2264	.04026	1288	.2409	.2295	.0388
4	.4000	.2270	.04047	1295	.2425	.2295	.0409
5	.4002	.2264	.04026	1288	.2413	.2295	.0401
6	.3998	.2264	.04026	1288	.2410	.2295	.0391
7	.4005	.2262	.04018	1286	.2410	.2295	.0400
8	.4002	.2264	.04026	1288	.2414	.2295	.0402
9	.4000	.2264	.04026	1288	.2414	.2295	.0404
10	.3997	.2260	.04012	1284	.2415	.2295	.0403
Average	.....	.....	.....	.....	.....	.....	.0400

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TABLE 4.—Experiments on Repeated Application of Same Load—Continued

No.	$L-L_2$	$L-L_3$	$L-L_4$	$L-L_5$	D_6 maximum	D_6 minimum	$L-L_6$
	Inch	Inch	Inch	Inch	Inch	Inch	Inch
1	0.0425	0.0450	0.0481	0.0492	0.2442	0.2345	0.0510
2	.0407	.0430	.0445	.0454	.2430	.2345	.0472
3	.0407	.0438	.0458	.0469	.2438	.2345	.0488
4	.0430	.0444	.0458	.0473	.2447	.2345	.0480
5	.0420	.0434	.0445	.0460	.2435	.2345	.0474
6	.0418	.0445	.0455	.0469	.2435	.2345	.0477
7	.0417	.0432	.0446	.0472	.2438	.2345	.0489
8	.0426	.0441	.0461	.0482	.2440	.2345	.0494
9	.0430	0456	.0468	.0482	.2430	.2345	.0497
10	.0423	.0450	.0469	.0480	.2442	.2345	.0498
Average	.0420	.0442	.0459	.0473	.....	.....	.0488

TABLE 5.—Experiments on Repeated Application of Same Load

No.	L	D	A	Load equivalent to 20 000 lbs./in.^{2}	D_1 maximum	D_1 minimum	$L-L_1$
	Inch	Inch	Inch^2	Pounds	Inch	Inch	Inch
1	0.4000	0.2256	0.03997	799	0.2315	0.2265	0.0165
2	.4001	.2257	.04001	800	.2319	.2265	.0161
3	.4000	.2267	.04036	807	.2332	.2270	.0172
4	.4000	.2257	.04001	800	.2328	.2260	.0185
5	.4002	.2263	.04022	804	.2332	.2265	0177
6	.4000	.2263	.04022	804	.2323	.2270	.0161
7	.3999	.2262	.04019	803	.2328	.2265	.0172
8	.4000	.2255	.03994	799	.2323	.2265	.0175
9	.4000	.2261	.04015	803	.2319	.2265	.0156
10	.4002	.2263	.04022	804	.2324	.2265	.0164
Average	.....	.....	.....	.....	.....	.....	.0169

No.	$L-L_2$	$L-L_3$	$L-L_4$	$L-L_5$	D_6 maximum	D_6 minimum	$L-L_6$
	Inch	Inch	Inch	Inch	Inch	Inch	Inch
1	0.0187	0.0196	0.0202	0.0207	0.2328	0.2264	0.0216
2	.0181	.0194	.0207	.0208	.2334	.2264	.0220
3	.0182	.0194	.0205	.0213	.2343	.2264	.0218
4	.0196	.....	.....	.....	.....	.....	.....
5	.0199	.0213	.0221	.0233	.2352	.2264	.0246
6	.0173	.0182	.0191	.0197	.2336	.2264	.0209
7	.0191	.0204	.0212	.0228	.2346	.2264	.0235
8	.0195	.0207	.0219	.0223	.2338	.2264	.0236
9	.0164	.0184	.0191	.0200	2341	.2264	.0205
10	.0175	.0188	.0205	.0214	.2340	.2264	.0226
Average	.0184	.0196	.0206	.0214	.....	.....	.0223

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NOTES ON TABLES 3, 4, AND 5.—These tables represent the results of experiments with cylinders of Series No. 2 by subjecting them to repeated compressions with single application of same load for about 2½ seconds.

The repeated loads were:

Table	Load
	Lbs./in.²
3.....	40 000
4.....	32 000
5.....	20 000

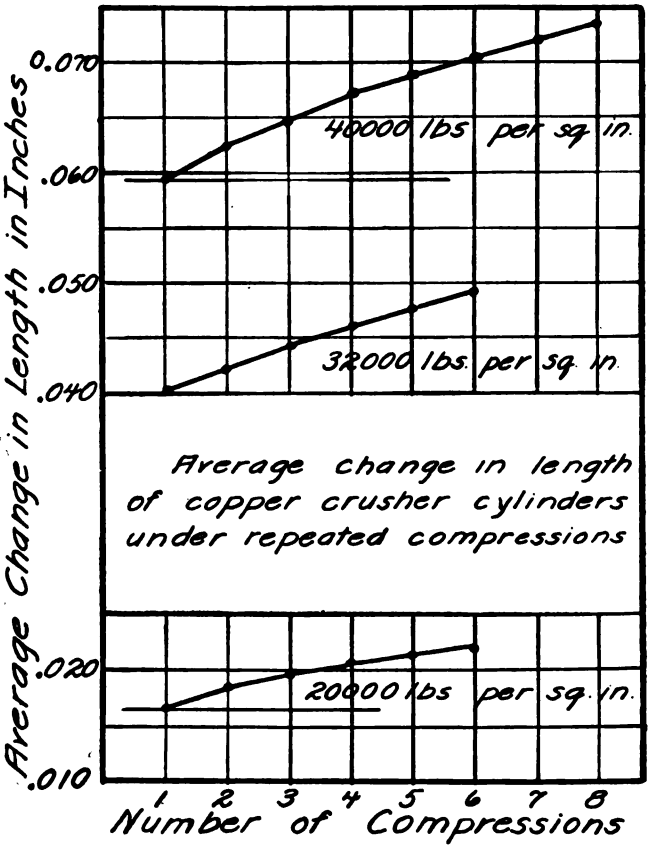

FIG. 5.—*Change of length of copper cylinders upon repeated compression*

All results are shown in Fig. 5.

The following conclusions may be drawn from these curves:

(1) The length of cylinders decreases considerably with the number of times the load is applied.

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(2) This change also decreases slightly with number of application of load as the hardness of copper increases.

(3) The slope of the curve representing results of repeated compression with the load P is steeper than slope of curve representing results of repeated compression with the load Q if load P is greater than Q.

(b) EXPERIMENTS ON EFFECT OF DURATION OF COMPRESSION APPLIED UNDER CONDITION (1)

TABLE 6.—Experiments on Effect of Duration of Compression Applied Under Condition (1)

L_1 = length of cylinders after applying pressure of 40 000 lbs./in.² for about 2½ seconds.

No.	L	D	A	Load equivalent to 40 000 lbs./in.²	$L-L_1$	No.	L	D	A	Load equivalent to 40 000 lbs./in.²	$L-L_1$
	Inch	Inch	Inch²	Lbs.	Inch		Inch	Inch	Inch²	Lbs.	Inch
1	0.3999	0.2258	0.04004	1601	0.0587	10	0.3999	0.2264	0.04026	1610	0.0583
2	.4001	.2262	.04018	1607	.0588	11	.4002	.2260	.04012	1604	.0589
3	.4002	.2266	.04033	1613	.0587	12	.4001	.2257	.04001	1600	.0579
4	.4000	.2257	.04001	1600	.0570	13	.3999	.2259	.04008	1603	.0609
5	.3997	.2263	.04022	1609	.....	14	.4000	.2264	.04026	1610	.0596
6	.3999	.2260	.04012	1604	.0579	15	.3999	.2257	.04001	1600	.0592
7	.3998	.2265	.04029	1612	.0591						
8	.4001	.2259	.04008	1603	.0589	Average	.....	.....	.....	.....	.0588
9	.3998	.2260	.04012	1604	.0601						

TABLE 7.—Experiments on the Effect of Duration of Compression Applied Under Condition (1)

L_1 = length of cylinders after applying pressure of 40 000 lbs./in.² for about 25 seconds.

No.	L	D	A	Load equivalant to 40 000 lbs./in.²	$L-L_1$
	Inch	Inch	Inch²	Pounds	Inch
1	0.3999	0.2256	0.03997	1598	0.0601
2	.4002	.2264	.04026	1610	.....
3	.3999	.2262	.04018	1607	.....
4	.4002	.2265	.04029	1612	.0594
5	.3999	.2262	.04018	1607	.0609
6	.3999	.2261	.04015	1606	.0600
7	.3998	.2256	.03997	1598	.0604
8	.4001	.2263	.04022	1609	.0603.
9	.3999	.2267	.04036	1614	.0600
10	.4000	.2262	.04018	1607	.0579
11	.4002	.2257	.04001	1600	.0580
12	.4002	.2257	.04001	1600	.0608
13	.4000	.2264	.04026	1610	.0578
14	.3998	.2262	.04018	1607	.....
15	.3999	.2263	.04022	1609	.0602
16	.3998	.2261	.04015	1606	.0611
Average	.....	.....	.....	.....	.0598

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TABLE 8.—Experiments on Effect of Duration of Compression Applied Under Condition (1)

L_1—length of cylinders after applying pressure of 40 000 lbs./in.² for about 5 seconds.

No.	L	D	A	Load equivalent to 40 000 lbs./in.²	$L-L_1$
	Inch	Inch	Inch²	Pounds	Inch
1	0.3999	0.2266	0.04033	1613	0.0579
2	.4009	.2263	.04022	1609	.0600
3	.4002	.2256	.03997	1598	.0597
4	.4001	.2265	.04029	1612	.0593
5	.4001	.2260	.04012	1604	.0585
6	.3998	.2258	.04004	1601	.0583
7	.4002	.2256	.03997	1598	.0592
8	.4002	.2265	.04029	1612	.0592
9	.3999	.2265	.04029	1612	.0595
10	.4000	.2256	.03997	1598	.0586
11	.4000	.2257	.04001	1600	.0593
12	.4000	.2267	.04036	1614	.....
13	.4002	.2257	.04001	1600	.0609
14	.3998	.2260	.04012	1604	.0595
15	.3999	.2259	.04008	1603	.0609
Average	.....	.....	.....	.....	.0593

NOTES ON TABLES 6, 7, AND 8.—In this case experiments were made upon the cylinders of Series No. 2. Experiments consisted of compressions of 40 000 pounds per square inch applied under condition (1) for different periods, 2½, 5, and 25 seconds. Average changes in length are 0.0588, 0.0593, and 0.0598 inch, respectively. These experiments show slight increase of total set with increased time of application of load.

(c) EXPERIMENTS ON EFFECT OF TEMPERATURE OF ANNEALING

TABLE 9.—Experiments on Effect of Temperature of Annealing

[These copper cylinders were annealed at 1200° F (650° C).]

No.	L	D	A	Load equivalent to 30 000 lbs./in.²	D_1 maximum	$L-L_1$	Load equivalent to 35 000 lbs./in.²	$L-L_2$	Load equivalent to 40 000 lbs./in.²	$L-L_3$
	Inch	Inch	Inch²	Pounds	Inch	Inch	Pounds	Inch	Pounds	Inch
1	0.5002	0.2223	0.03881	1164	0.2355	0.0478	1358	0.0628	1552	0.0774
2	.5000	.2225	.03888	1166	.2356	.0478	1361	.0626	1555	.0780
3	.5000	.2226	.03892	1168	.2359	.0482	1362	.0624	1557	.0775
4	.5000	.2215	.03853	1156	.2342	.0473	1349	.0614	1541	.0764
5	.5003	.2221	.03874	1162	.2351	.0476	1356	.0624	1550	.0772
6	.5000	.2226	.03892	1168	.2357	.0487	1362	.0626	1557	.0783
7	.5002	.2231	.03909	1173	.2360	.0472	1368	.0610	1564	.0772
8	.5001	.2228	.03899	1170	.2361	.0480	1365	.0627	1560	.0774
9	.5001	.2225	.03888	1166	.2359	.0489	1361	.0627	1555	.0786
10	.4999	.2225	.03888	1166	.2355	.0477	1361	.0621	1555	.0772
11	.5002	.2218	.03864	1159	.2345	.0476	1352	.0616	1546	.0769
12	.5000	.2222	.03878	1163	.2353	.0486	1357	.0630	1551	.0783
Average	.....	.....	.....	.....	.....	.0480	.....	.0623	.....	.0775

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TABLE 10.—Experiments on Effect of Temperature of Annealing

[These cylinders were annealed at 1000° F (540° C)]

No.	L	D	A	Load equivalent to 30 000 lbs./in.²	$L-L_1$	D_1 maximum	Load equivalent to 35 000 lbs./in.²	$L-L_2$	Load equivalent to 40 000 lbs./in.²	$L-L_3$
	Inch	Inch	Inch²	Pounds	Inch	Inch	Pounds	Inch	Pounds	Inch
1	0.5003	0.2249	0.03973	1192	0.0456	0.2382	1391	0.0597	1589	0.0749
2	.5005	.2248	.03969	1191	.0465	.2378	1389	.0603	1588	.0756
3	.5010	.2255	.03994	1198	.0446	.2378	1398	.0589	1598	.0737
4	.5007	.2250	.03976	1193	.0464	.2381	1392	.0603	1590	.0756
5	.5007	.2257	.04001	1200	.0466	.2386	1400	.0609	1600	.0765
6	.5004	.2252	.03983	1195	.0458	.2382	1394	.0609	1593	.0748
7	.5004	.2250	.03976	1193	.0465	.3281	1392	.0606	1590	.0763
8	.5003	.2253	.03987	1196	.0450	.2379	1395	.0595	1595	.0733
9	.5004	.2253	.03987	1196	.0458	.2379	1395	.0601	1595	.0746
10	.5002	.2251	.03980	1194	.0461	.2378	1393	.0599	1592	.0748
11	.5005	.2254	.03990	1197	.0460	.2382	1397	.0604	1596	.0751
12	.5002	.2254	.03990	1197	.0454	.2379	1397	.0593	1596	.0744
Average					.0459			.0601		.0750

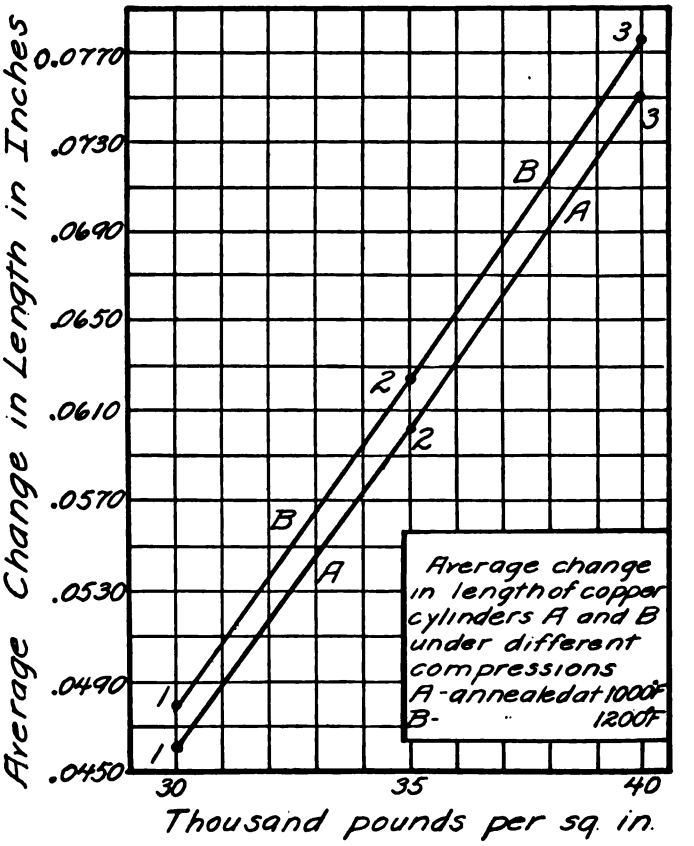

FIG. 6.—*Effect of annealing upon the change of length of copper cylinders upon compression*

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NOTES ON TABLES 9 AND 10.—These tables represent experiments with copper cylinders of Series No. 3, annealed at 1000 and 1200° F (540 and 650° C). Different compressions (30 000, 35 000, and 40 000 pounds per square inch) were made keeping the load under condition (1) for about 2½ seconds. Fig. 6 indicates that copper cylinders annealed at 1200° F are softer than those annealed at 1000° F.

(*d*) EXPERIMENTS ON INFLUENCE OF INITIAL PRECOMPRESSION WHEN LOADS SUCCESSIVELY APPLIED DIFFER CONSIDERABLY FROM EACH OTHER

TABLE 11.—Experiments on Influence of Initial Precompression when Loads Successively Applied Differ Considerably from Each Other

L_1 = length of cylinders after applying pressure of 20 000 lbs./in.² for about 2½ seconds.
L_2 = length of cylinders after applying pressure of 32 000 lbs./in.² for about 2½ seconds.
L_3 = length of cylinders after applying pressure of 40 000 lbs./in.² for about 2½ seconds.

No.	L	D	A	Load equivalent to 20 000 lbs./in.²	$L-L_1$	D_1 maximum	Load equivalent to 32 000 lbs./in.²	$L-L_2$	D_2 maximum	Load equivalent to 40 000 lbs./in.²	$L-L_3$	D_3 maximum
	Inch	Inch	Inch²	Lbs.	Inch	Inch	Lbs.	Inch	Inch	Lbs.	Inch	Inch
1	0.4000	0.2260	0.04012	802	0.0161	0.2322	1284	0.0395	0.2410	1605	0.0585	0.2481
2	.3998	.2264	.04026	805	.0168	.2331	1288	.0404	.2418	1610	.0596	.2494
3	.3998	.2257	.04001	800	.0175	.2327	1280	.0410	.2413	1600	.0607	.2494
4	.4000	.2259	.04008	802	.0180	.2323	1283	.0402	.2403	1603	.0595	.2487
5	.4000	.2253	.03987	797	.0156	.2318	1276	.0400	.2403	1595	.....	.....
6	.3998	.2261	.04015	803	.0154	.2322	1285	.0398	.2408	1606	.0594	.2490
7	.3998	.2260	.04012	802	.0158	.2317	1284	.0400	.2402	1605	.0601	.2486
8	.4000	.2260	.04012	802	.0150	.2317	1284	.0375	.2400	1605	.0560	.2476
9	.3999	.2263	.04022	804	.0160	.2322	1287	.0399	.2414	1609	.0586	.2488
10	.4001	.2263	.04022	804	.0164	.2327	1287	.0399	.2412	1609	.0589	.2490
11	.3998	.2265	.04029	806	.0175	.2333	1289	.0413	.2427	1612	.0603	.2498
12	.4000	.2265	.04029	806	.0167	.2326	1289	.0396	.2412	1612	.0588	.2488
13	.4000	.2258	.04004	801	.0171	.2322	1281	.0407	.2413	1602	.0607	.2494
14	.3999	.2263	.04022	804	.0176	.2331	1287	.0406	.2415	1609	.0598	.2493
15	.3998	.2268	.04040	808	.0158	.2332	1293	.0396	.2420	1616	.0590	.2498
Average	.....	.....	.....	.....	.0165	.....	.....	.0400	.....	.....	.0593	.....

NOTES ON TABLE 11.—Here are represented experiments with copper cylinders of Series No. 2. Successive compressions (20 000, 32 000, and 40 000 pounds per square inch) were made on same cylinders, keeping the load under condition (1) for about 2½ seconds.

Comparing the average changes in length of cylinders with those of other tables after the first compression, we can conclude that in this case change in length depends only on value of load of the last compression, no matter whether previous compressions had been made or not.

After the first compression of 32 000 pounds per square inch, Tables 4 and 12 show an average $L-L_1$ of 0.0400 inch. After two compressions, the first of which was 20 000 pounds per square inch and the last 32 000 pounds per square inch, Table No. 11 shows an average $L-L_2$ of 0.0400 inch.

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After first compression of 40 000 pounds per square inch:

Table No.	Results of first compression $L-L_1$
	Inch
6	0.0588
3	.0592
16	.0589
19	.0588

After three compressions, the first of which was 20 000 pounds per square inch, the second 32 000, and the third 40 000 pounds per square inch, Table 11 shows an average $L-L_3$ of 0.0593 inch. Comparing these figures—that is, 0.0400 with 0.0400 inch, and 0.0593 with 0.0588, 0.0592, 0.0589, and 0.0588 inch—we see that a very close concordance was obtained. It must be noted, however, that this statement may apply only when previous loads differ considerably from the last load, as was the case in these experiments. (The first load was 20 000, the second load 32 000, and the last load was 40 000 pounds per square inch.)

(e) EXPERIMENTS ON EFFECT OF INITIAL PRECOMPRESSION WHEN DIFFERENCES BETWEEN LOADS SUCCESSIVELY APPLIED ARE ONLY SLIGHT

TABLE 12.—Experiments on the Effect of Initial Precompression when the Differences Between the Loads Successively Applied are only Slight

L_1 = length of cylinders after pressure of 32 000 lbs./in.² for about 2½ seconds.
L_2 = length of cylinders after pressure of 36 000 lbs./in.² for about 2½ seconds.
L_3 = length of cylinders after pressure of 38 000 lbs./in.² for about 2½ seconds.
L_4 = length of cylinders after pressure of 40 000 lbs./in.² for about 2½ seconds.

No.	L	D	A	Load equivalent to 32 000 lbs./in.²	$L-L_1$	D_1 maximum	Load equivalent to 36 000 lbs./in.²
	Inch	Inch	Inch²	Pounds	Inch	Inch	Pounds
1	0.3998	0.2258	0.04004	1281			
2	.3999	.2262	.04018	1286	0.0403	0.2410	1446
3	.3999	.2257	.04001	1280	.0405	.2410	1440
4	.3999	.2261	.04015	1285	.0384	.2411	1445
5	.3997	.2258	.04004	1281	.0386	.2407	1441
6	.3997	.2267	.04036	1292	.0407	.2425	1453
7	.3999	.2267	.04036	1292	.0409	.2422	1453
8	.3999	.2263	.04022	1287	.0391	.2415	1448
9	.3999	.2265	.04029	1289	.0392	.2417	1450
10	.3999	.2267	.04036	1292	.0409	.2423	1453
11	.3998	.2267	.04036	1292	.0408	.2425	1453
12	.3997	.2260	.04012	1284	.0399	.2413	1444
13	.3995	.2259	.04008	1283	.0398	.2410	1443
14	.3996	.2263	.04022	1287	.0408	.2412	1448
15	.3997	.2268	.04040	1293			
Average					.0400		

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TABLE 12.—Experiments on the Effect of Initial Precompression when the Differences Between the Loads Successively Applied are only Slight—Continued

No.	$L-L_2$	Load equivalent to 38 000 lbs./in.²	$L-L_3$	Load equivalent to 40 000 lbs./in.²	$L-L_4$	Load D_4 maximum
	Inch	Pounds	Inch	Pounds	Inch	Inch
1						
2	0.0496	1527	0.0570	1607	0.0624	0.2486
3	.0502	1520	.0561	1600	.0624	.2495
4	.0496	1526	.0553	1606	.0612	.2493
5	.0495	1522	.0550	1602	.0607	.2491
6	.0502	1534	.0573	1614	.0627	.2496
7	.0497	1534	.0557	1614	.0618	.2498
8	.0499	1528	.0560	1609	.0616	.2494
9	.0505	1531	.0563	1612	.0624	.2494
10	.0506	1534	.0568	1614	.0629	.2500
11	.0509	1534	.0575	1614	.0638	.2508
12	.0495	1525	.0551	1605	.0618	.2488
13	.0497	1523	.0553	1603	.0609	.2489
14	.0504	1528	.0566	1609	.0620	.2494
15						
Average	.0500		.0562		.0620	

TABLE 13.—Experiments on Effect of Initial Precompression when Differences Between Loads Successively Applied are only Slight

L_1=length of cylinders after applying pressure of 36 000 lbs./in.² for about 2½ seconds.
L_2=length of cylinders after applying pressure of 38 000 lbs./in.² for about 2½ seconds.
L_3=length of cylinders after applying pressure of 40 000 lbs./in.² for about 2½ seconds.

No.	L	D	A	Load equivalent to 36 000 lbs./in.²	$L-L_1$	D_1 maximum	Load equivalent to 38 000 lbs./in.²	$L-L_2$	Load equivalent to 40 000 lbs./in.²	$L-L_3$	D_3 maximum
	Inch	Inch	Inch²	Pounds	Inch	Inch	Pounds	Inch	Pounds	Inch	Inch
1	0.3995	0.2260	0.04012	1444	0.0488	0.2447	1525	0.0545	1605	0.0610	0.2489
2	.3995	.2260	.04012	1444	.0486	.2445	1525	.0550	1605	.0601	.2488
3	.3994	.2263	.04022	1448	.0483	.2448	1528	.0549	1609	.0607	.2490
4	.3997	.2268	.04040	1454	.0500	.2462	1535	.0559	1616	.0622	.2506
5	.3997	.2262	.04018	1446	.0488	.2448	1527	.0557	1607	.0615	.2492
6	.3998	.2266	.04033	1452	.0501	.2451	1533	.0562	1613	.....	.....
7	.3994	.2257	.04001	1440	.0483	.2442	1520	.0553	1600	.0609	.2489
8	.3994	.2267	.04036	1453	.0496	.2458	1534	.0561.	1614	.0619	.2501
9	.3998	.2254	.03990	1436	.2447	.2448	1516	.0555	1596	.0613	.2486
10	.3997	.2255	.03994	1438	.0500	.2452	1518	.0564	1598	.....	.....
11	.4000	.2265	.04029	1450	.0499	.2455	1531	.0565	1612	.0625	.2500
12	.3996	.2258	.04004	1441	.0480	.2445	1522	.0550	1602	.0604	.2486
13	.4000	.2257	.04001	1440	.0505	.2451	1520	.0563	1600	.0628	.2492
14	.3999	.2256	.03997	1439	.0485	.2440	1519	.0548	1599	.0606	.2483
15	.3998	.2258	.04004	1441	.0501	.2450	1522	.0563	1602	.0625	.2495
Average	.....	.....	.....	.....	.0493	.....	.....	.0556	.....	.0614	.....

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TABLE 14.—Experiments on the Effect of Initial Precompression when the Differences Between the Loads Successively Applied are only Slight

L_1= length of cylinders after applying pressure of 38 000 lbs./in.² for about 2½ seconds.
L_2= length of cylinders after applying pressure of 40 000 lbs./in.² for about 2½ seconds.

No.	L	D	A	Load equivalent to 38 000 lbs./in.²	$L-L_1$	D_1 maximum	Load equivalent to 40 000 lbs./in.²	$L-L_2$	D_2 maximum
	Inch	Inch	Inch²	Lbs.	Inch	Inch	Lbs.	Inch	Inch
1	0.3998	0.2267	0.04036	1534	0.0550	0.3483	1614	0.0620	0.2503
2	.4000	.2260	.04012	1525	.0528	.3465	1605	.....	.....
3	.4001	.2262	.04018	1527	.0533	.3466	1607	.0597	.2487
4	.3999	.2255	.03994	1518	.0549	.3466	1598	.0616	.2490
5	.3999	.2256	.03997	1519	.0530	.3465	1599	.0595	.2489
6	.4000	.2262	.04018	1527	.0540	.3467	1607	.0609	.2494
7	.3998	.2257	.04001	1520	.0556	.3468	1600	.0621	.2492
8	.4001	.2262	.04018	1527	.0540	.3464	1607	.0604	.2489
9	.4001	.2259	.04008	1523	.0528	.3467	1603	.0599	.2489
10	.4002	.2264	.04026	1530	.0538	.3470	1610	.0598	.2496
11	.4000	.2257	.04001	1520	.0546	.3469	1600	.0608	.2486
12	.4000	.2264	.04026	1530	.0540	.3468	1610	.0607	.2496
13	.4001	.2256	.03997	1519	.0539	.3463	1599	.0597	.2486
14	.4002	.2260	.04012	1525	.....	.....	.....	.....	.....
15	.4003	.2263	.04022	1528	.0549	.3470	1609	.0608	.2492
Average	.....	.....	.....	.....	.0540	.....	.....	.0606	.....

TABLE 15.—Effect of Initial Precompression when Differences Between Loads Successively Applied are only Slight

Reference	32 000 lbs./in.²	36 000 lbs./in.²	38 000 lbs./in.²	40 000 lbs./in.²
See Tables Nos. 4, 11, and 12	$L-L_1=0.0400$	$L-L_2=0.0500$	$L-L_3=0.0562$	$L-L_4=0.0620$
See Table No. 13	.....	$L-L_1=0.0493$	$L-L_2=0.0556$	$L-L_3=0.0614$
See Table No. 14	.....	.....	$L-L_1=0.0540$	$L-L_2=0.0606$
See Tables Nos. 3, 6, 11, 16, and 19.	.....	.....	.....	$L-L_1=0.0590$

NOTES ON TABLES 12, 13, 14, AND 15.—Experiments were made upon cylinders of the same stock; that is, cylinders of the Series No. 2.

Successive compressions of 32 000, 36 000, 38 000, and 40 000 pounds per square inch were made under condition (1), keeping the load on for about 2½ seconds.

Average changes in length under initial and successive compressions are shown in Table 15. Here are collected from Tables 3, 4, 6, 11, 12, 13, 14, 16, and 19 average changes of length (total sets). These data show the difference between the change in length of cylinders which were compressed at load P and of those previously compressed using a load Q slightly lower than load P and then compressed at load P.

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It should be noted that in the case where several successive loads, differing by 2000 to 4000 pounds per square inch, had previously been applied the total change of length increases as the number of loads previously applied increases. All these results agree with experiments made with repeated compressions of same and smaller (within certain limits) loads applied on copper which is known to have practically no yield point.

(*f*) EXPERIMENTS ON EFFECT OF DURATION OF COMPRESSION APPLIED UNDER CONDITION (2) AND COMPARISON OF RESULTS OF DIFFERENT METHODS OF LOADING

TABLE 16.—Experiments on Effect of Duration of Compression Applied Under Condition (2)

L_1 = length of cylinders after applying load of 40 000 lbs./in.² for about 2½ seconds.

No.	L	D	A	Load equivalent to 40 000 lbs./in.²	$L-L_1$	D_1 maximum
	Inch	Inch	Inch²	Pounds	Inch	Inch
1	0.4000	0.2261	0.04015	1606	0.0582	0.2485
2	.4004	.2259	.04008	1603	.0598	.2488
3	.4000	.2259	.04008	1603	.0587	.2487
4						
5	.4003	.2259	.04008	1603	.0586	.2482
6	.3997	.2262	.04018	1607	.0579	.2484
7	.4005	.2265	.04029	1612	.0588	.2486
8	.4000	.2260	.04012	1604	.0588	.2488
9	.4000	.2266	.04033	1613	.0594	.2496
10	.4001	.2256	.03997	1599	.0601	.2489
Average					.0589	

TABLE 17.—Experiments on Effect of Duration of Compression Applied Under Condition (2)

L_1 = length of cylinders after applying pressure of 40 000 lbs./in.², holding beam balanced for about 5 seconds.

No.	L	D	A	Load equivalent to 40 000 lbs./in.²	$L-L_1$	D_1 maximum
	Inch	Inch	Inch²	Pounds	Inch	Inch
1	0.3996	0.2263	0.04022	1609		0.2494
2	.3996	.2266	.04033	1613	.0589	.2497
3	.3995	.2263	.04022	1609	.0598	.2494
4	.3996	.2257	.04001	1600	.0611	.2494
5	.3995	.2268	.04040	1616	.0596	.2505
6	.3995	.2265	.04029	1612	.0594	.2495
7	.3996	.2259	.04008	1603		.2475
8	.3996	.2253	.03987	1595	.0596	.2490
9	.3996	.2263	.04022	1609		.2503
10	.3995	.2263	.04022	1609	.0590	.2493
11	.4002	.2258	.04004	1602	.0604	.2486
12	.4000	.2255	.03994	1598	.0594	.2487
Average					.0597	

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TABLE 18.—Experiments on Effect of Duration of Compression Applied Under Condition (2)

L_1 = length of cylinders after applying pressure of 40 000 lbs./in.,² holding beam balanced for about 25 seconds.

	L	D	A	Load equivalent to 40 000 lbs./in.²	$L-L_1$	D_1 maximum
	Inch	Inch	Inch²	Pounds	Inch	Inch
1	0.4007	0.2262	0.04018	1607	0.0615	0.2495
2	.4003	.2261	.04015	1606	.0591	.2489
3	.4005	.2257	.04001	1600	.0590	.2485
4	.4003	.2262	.04018	1607	.0608	.2494
5	.4006	.2266	.04033	1613	.0609	.2505
6	.4006	.2257	.04001	1600	.0604	.2494
7	.4005	.2256	.03997	1599	.0613	.2488
8	.4005	.2258	.04004	1602	.0615	.2495
9	.4004	.2255	.03994	1598	.0611	.2486
10	.4008	.2263	.04022	1609	.0590	.2487
11	.4008	.2265	.04029	1612	.0611	.2495
12	.4008	.2264	.04026	1610	.0601	.2492
Average					.0605	

NOTES ON TABLES 16, 17, AND 18.—These tables represent the results of experiments made with cylinders of same series, No. 2, by subjecting them to compressions of 40 000 pounds per square inch applied under condition (2) for different periods—2½, 5, and 25 seconds—that is, holding beam balanced for these different periods.

In order to keep the beam balanced in this case only one additional application of same load (impulse) was needed after the beam had started to drop. Average changes in length are 0.0589, 0.0597, and 0.0605 inch, respectively.

It may be of interest to compare these results with those obtained in the experiments where the same load, 40 000 pounds per square inch, was held for different periods after the balance had been obtained and the beam allowed to drop, and also with those results obtained where the same load of 40 000 pounds per square inch was applied successively two times (for 2½ seconds each time), the second load being applied at some interval after the first load was entirely released.

1. In the case when load of 40 000 pounds per square inch was held for different periods after balance was obtained, the beam being allowed to drop (Tables 6, 7, and 8), the decrease in length of cylinders as compared with the length after compression for

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2½ seconds (considering average change for 2½ seconds is equal to 0.0590 inch) is shown here:

Time of compression	Difference in decrease
Seconds	Inch
2½	0.0000
5	.0003
25	.0008

2. In case where same load of 40 000 pounds per square inch was held for different periods after balance was obtained, but holding beam balanced by means of one additional application of same load (Tables 16, 17, and 18), these differences were:

Time of compression	Difference in decrease
Seconds	Inch
2½	0.0000
5	.0007
25	.0015

3. In the case of one repeated application of the same load of 40 000 pounds per square inch (Table 4) the difference between the decrease in length after first and second applications for 2½ seconds each is 0.0626—0.0590 inch = 0.0036 inch. Hence it may be seen: (*a*) That the application of the same load for the same period (greater than 3 seconds), but made under condition (2)—that is, holding the beam balanced by means of one additional application (impulse) of the same load—causes about twice the decrease in length observed in the case when the beam is allowed to drop after the balance is obtained; (*b*) that where the load is applied twice (for 2½ seconds each time), the second load being applied at some intervals after the first load has been entirely released, the decrease in length is much greater than in the first two cases, even when the load is held there as long as 25 seconds.

(*g*) EXPERIMENTS ON AGING OF COPPER CYLINDERS OF SERIES No. 2, TABLES 19-25

The duration of the compression in these experiments was about 2½ seconds.

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TABLE 19.—Compression Tests Relating to Experiments on Aging of Copper Cylinders

L_1 = length of cylinders after applying pressure of 40 000 lbs./in.²

No.	L	D	A	Load equivalent to 40 000 lbs./in.²	$L-L_1$
	Inch	Inch	Inch²	Pounds	Inch
1	0.4001	0.2266	0.04033	1613	0.0599
2	.4002	.2259	.04008	1603	.0587
3	.4001	.2265	.04029	1612	.0596
4	.4001	.2258	.04004	1602	.0611
5	.4000	.2258	.04004	1602	.0592
6	.4001	.2259	.04008	1603	.0558
7	.4000	.2253	.03987	1595	.0618
8	.4000	.2255	.03994	1598	.0555
9	.4002	.2260	.04012	1605	.0592
10	.4003	.2265	.04029	1612	.0606
11	.4001	.2262	.04018	1607	.0578
12	.4002	.2264	.04026	1610	.0591
13	.4001	.2257	.04001	1600	.0608
14	.4004	.2258	.04004	1602	.0598
15	.4003	.2266	.04033	1613	.0579
Average					.0588

NOTES ON TABLE 19.—The table shows the mean total set of cylinders compressed at 40 000 pounds per square inch. The total set was 0.0599 inch.

TABLE 20.—Compression Tests Relating to Experiments on Aging of Copper Cylinders

L_1 = length of cylinders after applying a pressure of 38 000 lbs./in.² for about 2½ seconds.
L_2 = length of cylinders after applying a pressure of 40 000 lbs./in.² for about 2½ seconds, the pressure of 40 000 lbs./in.² being applied 1 hour after the pressure of 38 000 lbs.

No.	L	D	A	Load equivalent to 38 000 lbs./in.²	$L-L_1$	Load equivalent to 40 000 lbs./in.²	$L-L_2$
	Inch	Inch	Inch²	Pounds	Inch	Pounds	Inch
1	0.4002	0.2260	0.04012	1525	0.0530	1605	0.0612
2	.4002	.2261	.04015	1526	.0552	1606	.0622
3	.4004	.2257	.04001	1520	.0556	1600	.0619
4	.4001	.2265	.04029	1531	.0541	1612	.0616
5	.4001	.2260	.04012	1525	.0554	1605	.0616
6	.4005	.2262	.04018	1527	.0539	1607	.0594
7	.4001	.2258	.04004	1522	.0551	1602	.0608
8	.4003	.2263	.04022	1528	.0529	1609	.0555
9	.4005	.2263	.04022	1528	.0547	1609	.0612
10	.4001	.2259	.04008	1523	.0514	1603	.....
11	.4002	.2261	.04015	1526	.0517	1606	.0588
12	.4001	.2257	.04001	1520	.0545	1600	.0616
13	.4003	.2258	.04004	1522	.0513	1602	.....
14	.4003	.2258	.04004	1522	.0544	1602	.0606
15	.4005	.2257	.04001	1520	.0543	1600	.0607
Average					.0538		.0608

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NOTES ON TABLE 20.—The cylinders were precompressed at 38 000 pounds per square inch, and one hour later they were compressed at 40 000 pounds per square inch. Total set was 0.0608 inch.

TABLE 21.—Compression Tests Relating to Experiments on Aging of Copper Cylinders

L_1 = length of cylinders after applying a pressure of 38 000 lbs./in.² for about 2½ seconds
L_2 = length of cylinders after applying a pressure of 40 000 lbs./in.² for about 2½ seconds, the pressure of 40 000 lbs. being applied after aging cylinders at 100° C during 28 days.

No.	L	D	A	Load equivalent to 38 000 lbs./in.²	$L-L_1$	Load equivalent to 40 000 lbs./in.²	$L-L_2$
	Inch	Inch	Inch²	Pounds	Inch	Pounds	Inch
1	0.4005	0.2257	0.04001	1520	0.0551	1600	0.0632
2	.4003	.2258	.04004	1522	.0544	1602	.0626
3	.4006	.2257	.04001	1520	.0566	1600	.0627
4	.4004	.2257	.04001	1520	.0519	1600	.0596
5	.4005	.2263	.04022	1528	.0518	1609	.0582
6	.4001	.2263	.04022	1528	.0543	1609	.0618
7	.4002	.2264	.04026	1530	.0563	1610	.0641
8	.4003	.2258	.04004	1522	.0542	1602	.0614
9	.4003	.2264	.04026	1530	.0545	1610	.0627
10	.4003	.2265	.04029	1531	.0539	1612	.0618
11	.4005	.2255	.03994	1518	.0549	1598	.0628
12	.4000	.2262	.04018	1527	.0539	1607	.0619
13	.4004	.2270	.04047	1538	.0549	1619	.0623
14	.4000	.2258	.04004	1522	.0544	1602	.0615
15	.4002	.2260	.04012	1525	.0525	1605	.0615
Average					.0542		.0619

NOTES ON TABLE 21.—This table shows the results for copper cylinders, precompressed at 38 000 pounds per square inch, which had been held in an electric oven at 100° C for 28 days and then compressed at 40 000 pounds per square inch. The final total set in this case was 0.0619 inch.

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TABLE 22.—Compression Tests Relating to Experiments on Aging of Copper Cylinders

L_1 = length of cylinders after applying pressure of 38 000 lbs./in.² for about 2½ seconds.
L_2 = length of cylinders after applying pressure of 40 000 lbs./in.² for about 2½ seconds, the pressure of 40 000 lbs./in.² being applied after aging cylinders at 0° and 100° C alternately during 28 days.

No.	L	D	A	Load equivalent to 38 000 lbs./in.²	$L-L_1$	Load equivalent to 40 000 lbs./in.²	$L-L_2$
	Inch	Inch	Inch²	Pounds	Inch	Pounds	Inch
1	0.4004	0.2260	0.04012	1525	0.0558	1605	0.0636
2	.4001	.2259	.04008	1523	.0556	1603	.0630
3	.4001	.2260	.04012	1525	.0544	1605	.0624
4	.4004	.2263	.04022	1528	.0548	1609	.0613
5	.4001	.2261	.04015	1526	.0543	1606	.0624
6	.4002	.2260	.04012	1525	.0552	1605	.0637
7	.4000	.2259	.04008	1523	.0525	1603	.0593
8	.4001	.2268	.04040	1535	.0539	1616	.0626
9	.4003	.2258	.04004	1522	.0540	1602	.0624
10	.4002	.2267	.04036	1534	.0541	1614	.0618
11	.4002	.2259	.04008	1523	.0530	1603	.0614
12	.4000	.2267	.04036	1534	.0538	1614	.0609
13	.4005	.2254	.03990	1516	.0546	1596	.0622
14	.4000	.2257	.04001	1520	.0538	1600	.0615
15	.4005	.2261	.04015	1526	.0505	1606	.0575
Average					.0540		.0617

NOTES ON TABLE 22.—This table indicates that the final total set of 0.0617 inch was obtained on cylinders precompressed at 38 000 pounds per square inch and aged at 0 and 100° C, alternately, during 28 days and then compressed at 40 000 pounds per square inch.

TABLE 23.—Compression Tests Relating to the Experiments on Aging of Copper Cylinders

L_1 = length of cylinders after applying pressure of 38 000 lbs./in.² for about 2½ seconds.
L_2 = length of cylinders after applying pressure of 40 000 lbs./in.² for about 2½ seconds, the pressure of 40 000 lbs./in.² being applied after aging cylinders at room temperature for 30 days.

No.	L	D	A	Load equivalent to 38 000 lbs./in.²	$L-L_1$	Load equivalent to 40 000 lbs./in.²	$L-L_2$
	Inch	Inch	Inch²	Pounds	Inch	Pounds	Inch
1	0.4008	0.2258	0.04004	1522	0.0506	1602	0.0537
2	.4008	.2257	.04001	1520	.0561	1600	.0636
3	.4005	.2257	.04001	1520	.0558	1600	.0641
4	.4007	.2260	.04012	1525	.0560	1605	.0617
5	.4007	.2262	.04018	1527	.0550	1607	.0612
6	.4008	.2259	.04008	1523	.0550	1603	.0628
7	.4008	.2260	.04012	1525	.0537	1605	.0613
8	.4005	.2259	.04008	1523	.0527	1603	.0602
9	.4010	.2260	.04012	1525	.0552	1605	.0616
10	.4008	.2259	.04008	1523	.0555	1603	.0624
11	.4011	.2262	.04018	1527	.0553	1607	.0621
12	.4006	.2267	.04036	1534	.0547	1614	.0618
Average					.0546		.0614

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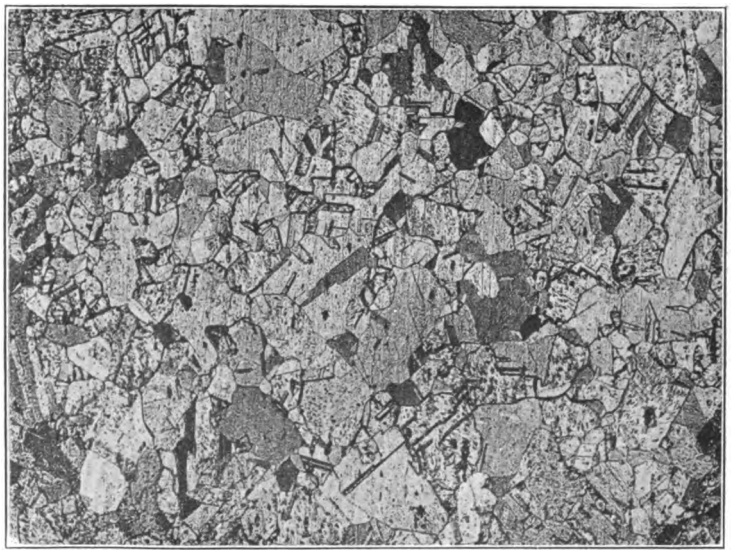

FIG. 7.—*Longitudinal section of copper cylinder annealed at 650° C.* × *100*

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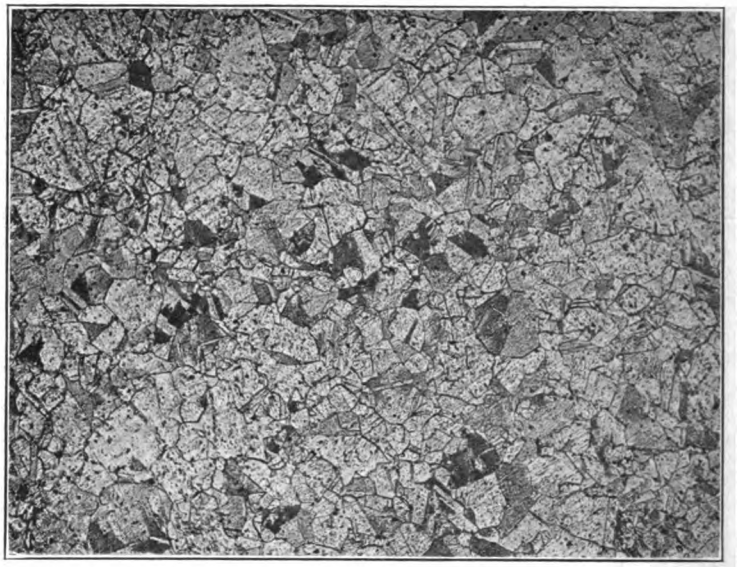

FIG. 8.—*Cross section of copper cylinder annealed at 650° C.* × 100

FIG. 9.—*Longitudinal section of copper cylinder annealed at 650° C.* × 100

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TABLE 24.—Compression Tests Relating to the Experiments on Aging of Copper Cylinders

L_1= length of cylinders after applying pressure of 38 000 lbs./in.² for about 2½ seconds.
L_2= length of cylinders after applying pressure of 40 000 lbs./in.² for about 2½ seconds, the pressure of 40 000 lbs./in.² being applied after aging cylinders at room temperature for 27 days.

No.	L	C	A	Load equivalent to 38 000 lbs./in.²	$L-L_1$	Load equivalent to 40 000 lbs./in.²	$L-L_2$
	Inch	Inch	Inch²	Pounds	Inch	Pounds	Inch
1	0.4000	0.2268	0.04040	1535	0.0547	1616	0.0630
2	.4001	.2260	.04012	1525	.....	.....	.....
3	.3999	.2260	.04012	1525	.0525	1605	.0592
4	.3996	.2258	.04004	1522	.0550	1602	.0621
5	.3996	.2265	.04029	1531	.0529	1612	.0609
6	.4001	.2259	.04008	1523	.0524	1603	.0588
7	.4001	.2257	.04001	1520	.0551	1600	.0626
8	.4001	.2256	.03997	1519	.0544	1599	.0618
9	.3996	.2258	.04004	1522	.0561	1602	.0618
10	.4002	.2260	.04012	1525	.0547	1605	.0621
11	.3999	.2255	.03994	1518	.0541	1598	.0621
12	.3998	.2261	.04015	1526	.0548	1606	.0628
Average	.....	.....	.....	.....	.0542	.....	.0616

NOTE ON TABLES 23 AND 24.—Cylinders precompressed at 38 000 pounds per square inch were aged at room temperature for 30 days and then compressed at 40 000 pounds per square inch. The mean total sets were 0.0614 and 0.0616 inch, respectively.

Intercomparing all results stated above (Table 25) it may be concluded that aging at temperatures of 0–100° C makes the compressed copper softer. It should be noted, however, that in the author's opinion the data in regard to aging is not sufficient; such an important problem needs very careful additional experiments in static as well as impact and firing tests after different periods of aging. The above results also are contradictory to those of H. W. R. Mason,[12] who found from his impact experiments that in copper crushers spontaneous annealing does not take place.

[12] H. W. R. Mason, Resistance of Copper Crushers during Compression, Arms and Explosives; July 1, 1918.

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TABLE 25.—Table Showing Average Results on Effect of Aging on Reduction in Length of Compressed Copper Cylinders

Reference to cylinders	Precompression at 38 000 lbs./in.²		Compression at 40 000 lbs./in.²	
	Date of precompression	Average total set of cylinders per inch	Date of compression	Average total set of cylinders per inch
		Inch		Inch
Table 14	June 26, 1919	0.0540	One hour later of same day, June 26, 1919.	0.0606
Table 20	Aug. 29, 1919	.0538	One hour later of same day, Aug. 29, 1919.	.0608
Table 21	July 30, 1919	.0542	Aug. 27, 1919, after aging at 100° C from July 30 to Aug. 27.	.0619
Table 22	do	.0540	Aug. 27, 1919, after aging at 0° C and 100° C alternately from July 30 to Aug. 27.	.0617
Table 23	do	.0546	Aug. 29, 1919, after aging at room temperature from July 30 to Aug. 29.	.0614
Table 24	July 31, 1919	.0542	Aug. 27, 1919, after aging at room temperature from July 31 to Aug. 27.	.0616

III. MICROSCOPIC EXAMINATION

Longitudinal and cross sections of copper cylinders were examined microscopically (Figs. 7–15). In all cases the cylinders were etched with NH_4OH and H_2O_2.

In order to show distortion of grains under pressure, some micrographs were taken of the same spots before and after compression—that is, the cylinder was etched, the micrograph was taken, and the spot marked. Then the cylinder was compressed at 30 000 pounds per square inch, and a micrograph of the marked spot was taken again. This is shown in Figs. 12–15, where (Figs. 13 and 15) many slip bands may be seen.

IV. SUMMARY

Several conclusions may be drawn from these experiments, which only confirm the already known phenomena, as well as those expected from the known properties of copper:

1. The length of copper crusher cylinders decreases considerably under repeated compressions of the same load. The relation between this decrease in length and number of times load was applied is nearly proportional within certain limits.

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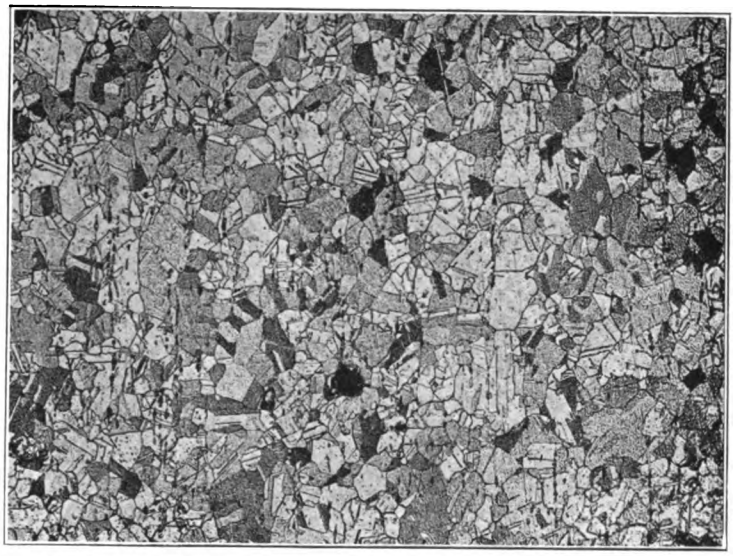

FIG. 10.—*Longitudinal section of copper cylinder annealed at 540° C.* × 100

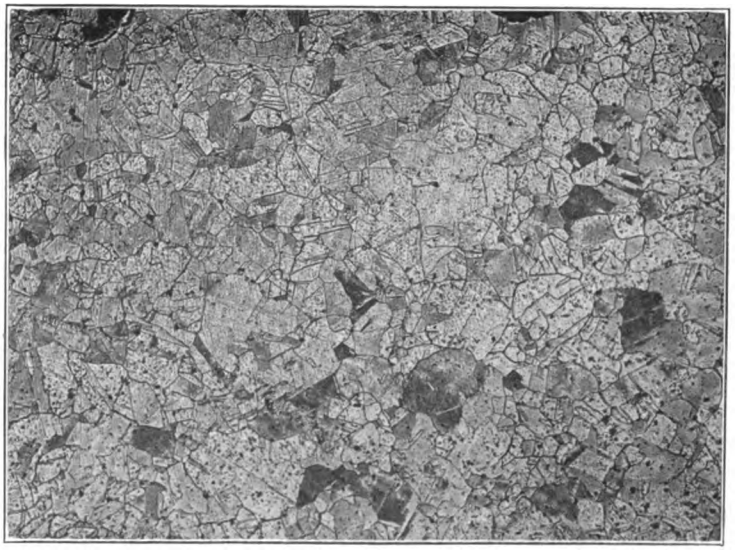

FIG. 11.—*Cross section of copper cylinder annealed at 540° C.* × 100

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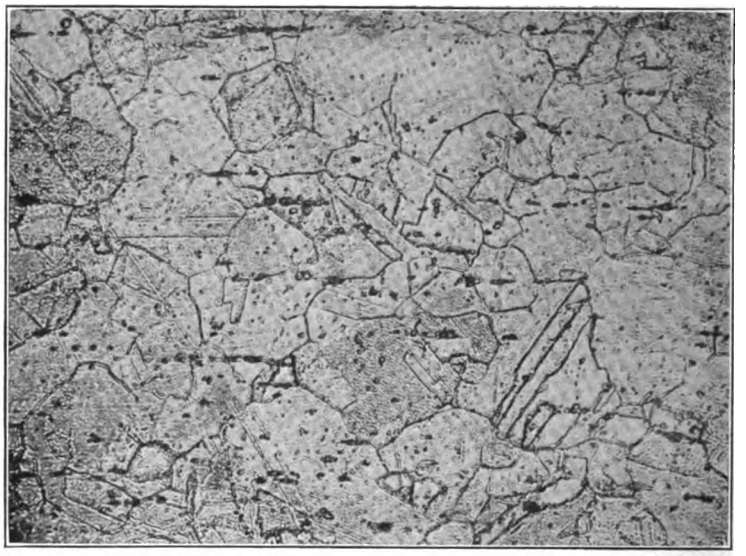

FIG. 12.—*Longitudinal section of copper cylinder annealed at 650° C. Micrograph represents the spot located midway bases and near to the axis of cylinder.* × 200

FIG. 13.—*Same cylinder and same spot as Fig. 12, but after compression at 30 000 lbs. per square inch.* × 200

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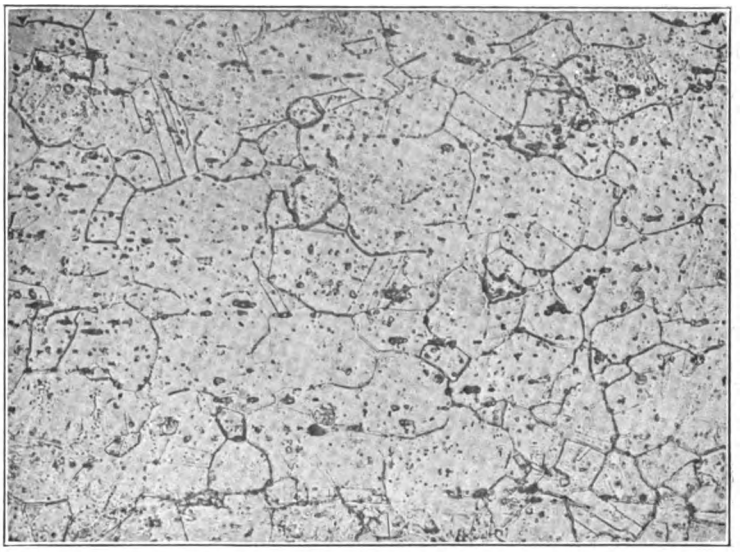

FIG. 14.—*Longitudinal section of copper cylinder annealed at 650° C. Spot is located near one of the bases of the cylinder and very near its periphery.* × 200

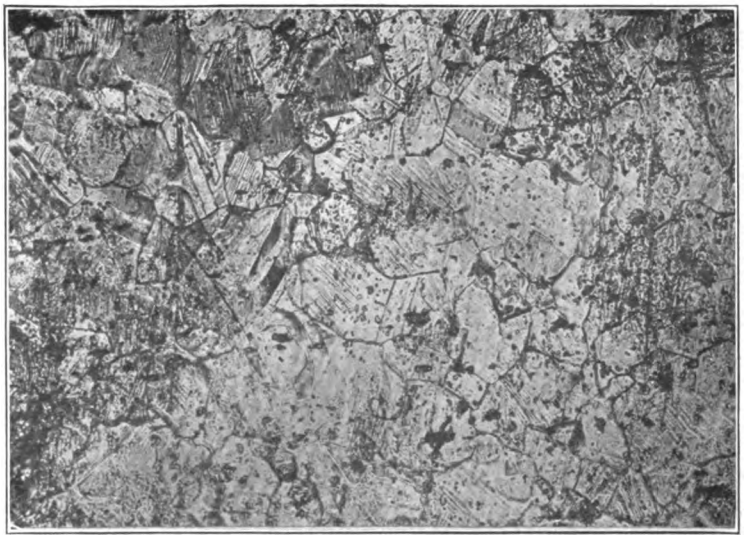

FIG. 15.—*Same cylinder and same spot as in Fig. 14, but after compression at 30 000 lbs. per square inch.* × 200

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2. The change in length with repeated constant loads is greater with the greater load.

3. Repeated application of successive decreasing loads causes a gradual decrease in length within certain limits.

4. The length changes but slightly with longer application of load when the load is applied in such a manner as to occasion the maximum stress only for an instant.

5. Application of the same load for the same period (greater than about 3 seconds), but holding the beam balanced by means of one additional application (impulse) of the same load after the beam had started to drop, causes about twice the decrease in length observed in the previous case, when the beam was allowed to drop after balance is obtained.

6. Double application of the same load for 2½ seconds each time, the second load being applied at some interval after the first load has been released, causes a decrease in length much greater than in the previous two cases. This holds true even when the pressure is applied for as long as 25 seconds in the case of paragraphs 4 and 5.

7. In case the last load is considerably greater than any previous loads, the change in length caused by the last load is practically independent of the previous loads—that is, it is the same as would be obtained by compressing a previously uncompressed cylinder.

8. When two successive loads of considerable amount (approximately 40 000 pounds per square inch) differing from one another by about 2 000 pounds per square inch, are applied, the second being greater than the first, the change in length due to the last load is considerably greater than that obtained where the pressure is applied on previously uncompressed cylinders, and this difference increases as the difference between the two loads successively applied decreases.

9. When several successive loads of considerable amount are applied, differing by about 2000 pounds per square inch, each greater than the preceding load, the total change in length of the cylinders due to the last compression increases with the number of loads previously applied.

10. Copper cylinders annealed at 1200° F 650° C are softer than those annealed at 1000° F 540° C.

11. It is probable that aging at temperatures within 0–100° C softens the compressed copper somewhat.

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12. Concerning the use of precompressed or uncompressed copper cylinders the conclusion from these experiments may be drawn for cases when the checking of every copper cylinder by precompression is necessary, a precompression of the cylinders at a pressure of at least 8000 pounds per square inch below the expected maximum pressure can be employed without impairing the ability of the precompressed cylinders to register the maximum pressure in equally reliable manner as an uncompressed cylinder would.

WASHINGTON, October 15, 1920.

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No. 186

OSCILLOGRAPH MEASUREMENTS OF THE INSTANTANEOUS VALUES OF CURRENT AND VOLTAGE IN THE BATTERY CIRCUIT OF AUTOMOBILES

BY

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Bureau of Standards

May 2, 1921

PRICE, 10 CENTS
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OSCILLOGRAPH MEASUREMENTS OF THE INSTANTANEOUS VALUES OF CURRENT AND VOLTAGE IN THE BATTERY CIRCUIT OF AUTOMOBILES

By G. W. Vinal and C. L. Snyder

ABSTRACT

This investigation was begun to study the demands upon starting and lighting batteries in the operation of various types of automobiles. By using an oscillograph, photographic records of the current and voltage were obtained. In addition to the data relative to the battery requirements, the interpretation of the records has indicated the possibility of using this method for the study of lubrication, and engine problems relating to torque, speed, friction, ignition, compression, and distributor action. Oscillograms obtained for five of the cars measured are shown in the illustrations. The results given are intended to be suggestive rather than quantitative measurements of performance.

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1. INTRODUCTION

For some months, at the request of the Motor Transport Corps of the War Department, the Bureau of Standards has been working on specifications for starting and lighting batteries. These specifications have now been completed and will be published as the appendix to a circular to be issued by this Bureau. Various laboratory tests have been made also on batteries of this class. In addition to these tests it seemed desirable to study the performance of batteries when in actual use on automobiles. Measurements of current and voltage of the batteries as made by the ordinary indicating instruments do not give accurate or adequate information because of the rapidly fluctuating values of these quantities. We have, therefore, measured the instantaneous values of current and voltage from oscillograph records. The

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variations of voltage and current when the engine is operated by the starter, and the variation of charging current through the battery under running conditions, are shown in the oscillograms that follow.

In the course of the experiments additional information was obtained with respect to the operation of the starter system and the engine itself. It is intended, therefore, in this paper to point out some of the effects relating to torque, temperature, compression, lubrication, distributor action, and flywheel velocity, in addition to the battery characteristics. The results which are given are suggestive only of the possible application of this method for the study of the performance of starter systems and internal-combustion engines. It is believed that a development of this method may provide an easy and exact method for studying the operation of various parts. This method was found entirely satisfactory for studying the problems connected with the battery.

Experiments have been made on a number of different cars. From the records obtained, we have chosen those of five cars which are designated in this paper as A, B, C, D, and E. Each of these cars was of a well-known make. For purposes of comparison, we have given in Table 1 data as to the history and characteristics of these cars which is essential to a proper understanding of the curves. All of the cars were in ordinary running condition at the time of the test.

TABLE 1.—Designation and Data of the Cars Tested

Designation of car	Year	Cylinders	Miles driven	Bore	Stroke	Starter system	Battery voltage
A	1916	4	15 000	3⅞	4½	Single unit	12
B	1920	4	5 000	3¾	4	Two unit	6
C	1917	6	12 000	3¼	4½	.....do	6
D	1917	4	20 000	3⅞	5	.....do	6
E	1919	8	18 000	3⅛	5⅛	Single unit	6

2. METHODS AND APPARATUS

The instantaneous values of current and voltage were obtained by means of a three-element moving-coil type oscillograph. In order to obtain records for periods of long duration, the ordinary film drum was replaced by a camera of special construction in which bromide paper in rolls 100 feet long was used. When

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making a record, the sensitized paper was wound upon a drum in the lower end of the camera after having passed the oscillograph slot through which the recording images were projected. One of the elements was used to record the voltage at the terminals of the battery, another the current through the battery circuit, and the third the time record, which consisted of the half-second ticks of a chronometer.

The camera was arranged so that the photographic paper moved at a uniform speed, irrespective of the amount of paper remaining in the camera. It was possible, however, to change the speed of the paper at will. Some of the records have been made with the paper moving at low speeds, about 2 cm per second, others at an intermediate speed of 3½ cm per second, but most of them at high speed, 7½ cm per second.

In order to measure the current through the battery circuit, a shunt of 0.006 of an ohm resistance was inserted in the circuit at the battery. Potential wires from this shunt were connected with the current element in the oscillograph. Measurements of the current and voltage were also made simultaneously by an ammeter and voltmeter. These meters have too slow a period to give the initial values. The meter readings, however, gave satisfactory average values for the current and voltage when the starter was operated at a uniform rate. Such readings were used as a check upon the calibration of the elements in the oscillograph. The deflections of the elements were approximately proportional to the current or voltage measured. In some cases, however, a deviation was found, which may have been due to the slight tension of the element. The tension was necessarily reduced to a minimum in order to give sufficient deflection. It was found necessary to calibrate the current and voltage elements with steady currents and voltages which could be accurately measured. This calibration was necessarily repeated each time the tension of any element was altered, or when the gold fuses were replaced. It will be noted that the half-second ticks of the time record are unequal. This was due to the action of the relay in the circuit. From the beginning of one second to the beginning of the next second, however, can be considered as one second to a very high degree of accuracy.

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3. VARIATIONS IN VOLTAGE AND CURRENT

The general character of the curves [1] obtained is shown in the figures following. All of these oscillograms begin at the right and are, therefore, to be read from right to left. A more detailed examination of these and similar figures will be given in the later

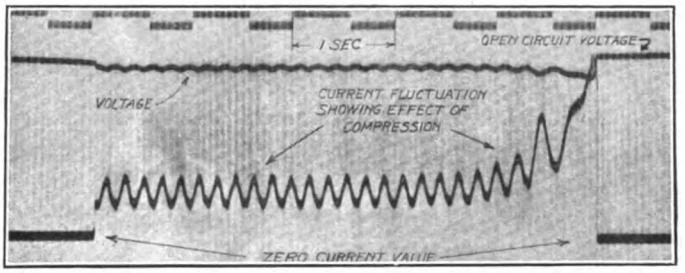

FIG. 1.—*Record of current and voltage in the battery circuit of car A when the starter was cranking the engine without ignition*

The fluctuations in these curves show the effect of compression in the cylinders. The initial current was approximately 125 amperes. The time scale in half-seconds is shown at the top of the record

sections. Fig. 1 shows curves for car A. The time intervals are recorded at the top of the record. The two curves below represent the fluctuations of the voltage and current of the battery when the starter is in operation. The successive peaks of the latter show the increase of current for each compressive stroke. Fig. 2 shows the operation of the same system with the ignition. It will be

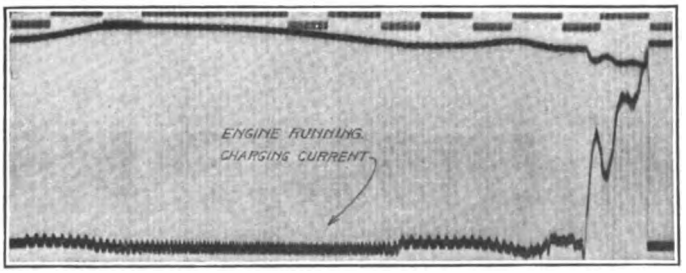

FIG. 2.—*Record of the same car as Fig. 1 when the engine was running*

The engine started on the second compression. The voltage curve falls at the beginning, but rises to higher values as soon as the charging current begins. The charging current lies slightly below the zero current value at the ends of the record

seen from the curve that the engine started on the second compression somewhat less than one second after the starter switch was closed. The variations in the current curve will be explained under the head of section 6. Figs. 3 and 4 show similar observa-

[1] Vincent, Trans. Soc. Auto. Eng., 11, part 1, p. 194; 1916 has derived mechanically the general shape of the torque curves for 6, 8, and 12 cylinder engines. The current curves of our oscillograms are proportional to the torque and show actual performance.

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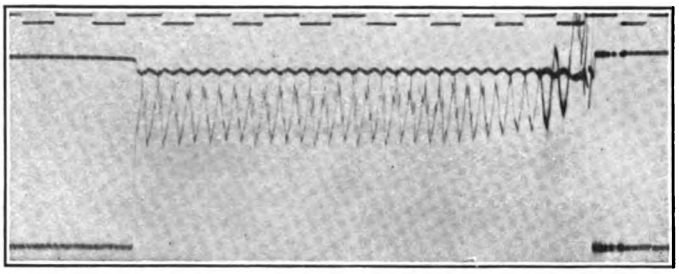

FIG. 3.—*Record of current and voltage in the battery circuit of car B operated without the ignition*

The current fluctuated rapidly during the first half-second through a range of more than 100 amperes, the maximum being about 300 amperes. The minimum values of voltage correspond to the maximum values of current

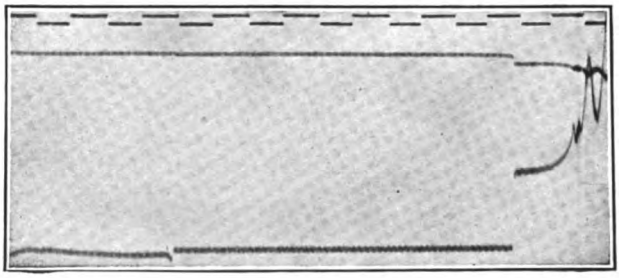

FIG. 4.—*Record of car B with engine starting*

The differences in operation between a two-unit and a single-unit system may be seen by comparing this figure with Fig. 2. The point at which the starter pedal was released can be seen

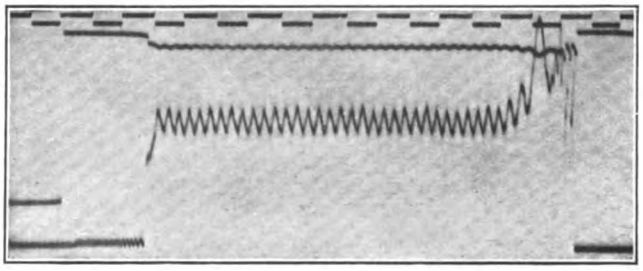

FIG. 5.—*Record of current and voltage in battery circuit of car C operated without the ignition*

The current fluctuated rapidly during the first half-second

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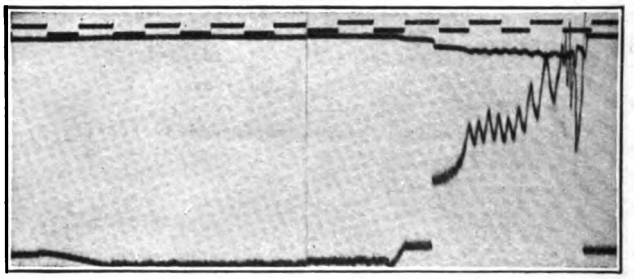

FIG. 6.—*Record of car C with ignition*

The engine started in about 2 seconds

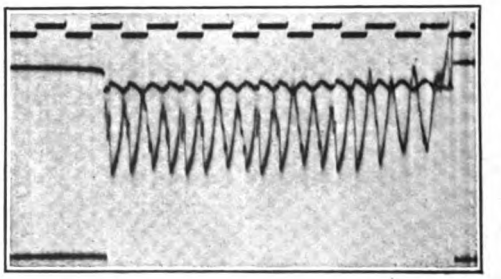

FIG. 7.—*Record of current and voltage in the battery circuit of car D operated without the ignition*

The initial current was about 250 amperes, the average running current about 120 amperes

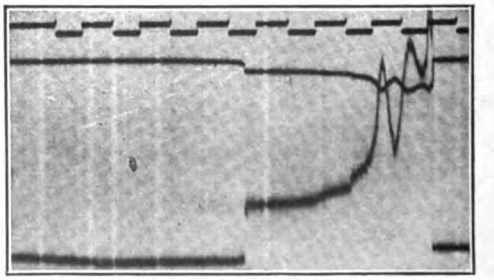

FIG. 8.—*Record for the same car as Fig. 7 with the ignition*

The engine started in about 1 second

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tions made on car B. The initial current values of both Figs. 3 and 4 are beyond the limits of the paper. During the first half second the current fluctuated violently between 200 and 300 amperes. In Fig. 3 it may be seen that the minimum voltage values correspond with the maximum current values. This is

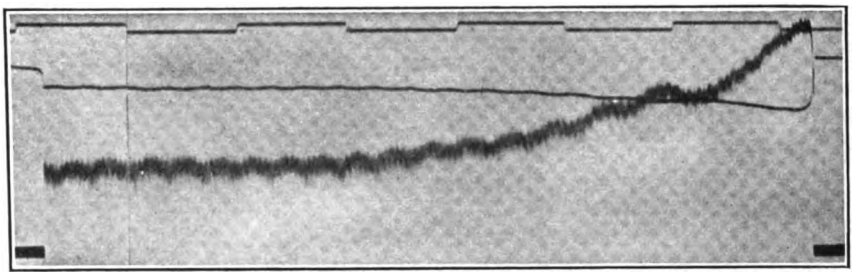

FIG. 9.—*Record of current and voltage in the battery circuit of car E as the starter cranks the engine, without the ignition*

The time scale of this and succeeding figures has been enlarged. The initial current was about 425 amperes. By comparing this figure with Fig. 18, in which only one bank of this engine was used, the overlapping effect of the cylinders may be seen

true of the other illustrations also, but in most of them it is not as clearly seen as here. The motor was started with the ignition on the battery in Fig. 4. The discontinuous point in the current curve shows the time at which the starter pedal was released. A corresponding rise in the voltage is to be observed. Figs. 5 and 6 were obtained on car C. The characteristics shown are somewhat

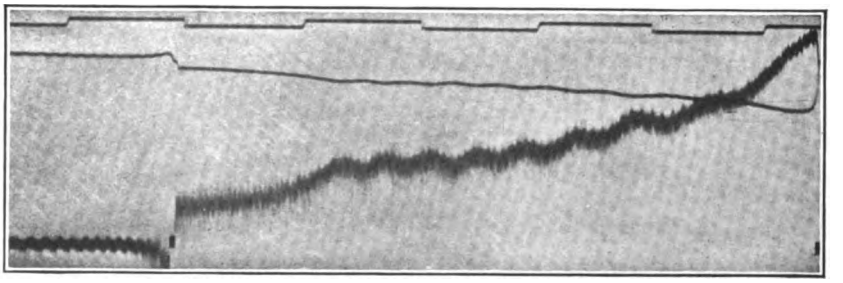

FIG. 10.—*Record similar to Fig. 9 but with the ignition*

A point of zero value in the current curve after the engine started indicates the operation of the reverse-current relay

similar to those in Figs. 3 and 4. Figs. 7 and 8 were obtained on car D. These curves are without the violent fluctuations observed during the first half second in the case of the four preceding curves. Figs. 9 and 10 show curves for car E. The initial current was approximately 425 amperes, decreasing to an average running value of about 150 amperes, as shown in Fig. 9. In Fig. 10 the

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ignition switch was closed and the engine started at the end of two and one-fourth seconds.

In the illustrations for the various cars, characteristic differences are apparent in the current and voltage curves. A comparison of these curves, shown in Figs. 1, 3, 5, 7, and 9, indicates that some are more nearly sine curves than others. These differences are probably characteristic of differences in the designs of the engines. By making a harmonic analysis of such curves, it might be possible to trace the effects of the timing of the valves and other factors. For such an analysis it would be desirable to increase the time scale as has been done in Figs. 11, 12, and 13. We have not attempted such a harmonic analysis in this paper.

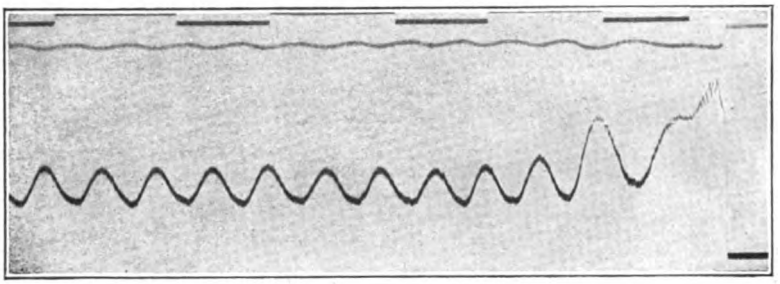

FIG. 11.—*Record from car A at radiator temperature 20° C*

The readings taken from these curves are given in Table 2

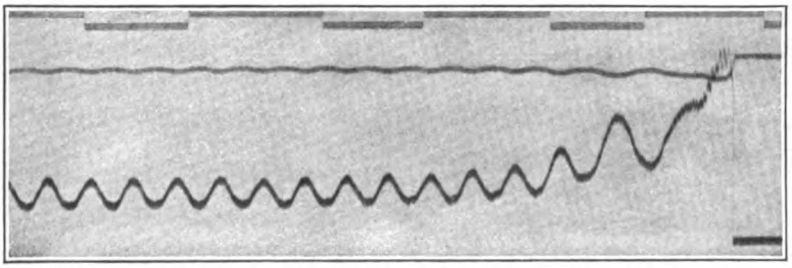

FIG. 12.—*Record as for Fig. 11 but at radiator temperature of 50° C*

The readings taken from these curves are given in Table 2

Figs. 2, 4, 6, 8, and 10 show characteristic differences in the starter systems, particularly between the single and two-unit systems.

The sensibility of the oscillograph was not the same for all of the measurements, and some differences in appearance of the curves have been caused by variations in the intensity and width of the beam of light. The cars were all in ordinary running condition.

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4. EFFECT OF TEMPERATURE

Figs. 11, 12, and 13 show the effect of the temperature of the engine on the current and voltage required for starting. The engine was operated by the starter without ignition. These curves were obtained on car A at radiator temperatures of 20, 50, and 77° C (68, 122, and 171° F), respectively. The initial values for current and voltage have been read from these curves and tabulated in Table 2, together with the values obtained when the speed was constant. The average operating values of current and voltage at constant speed as read from the curves agree closely with the values given by the indicating instruments in the circuit. The difference between the maximum and minimum values at constant speed is the amplitude of column 9 in the table. It will

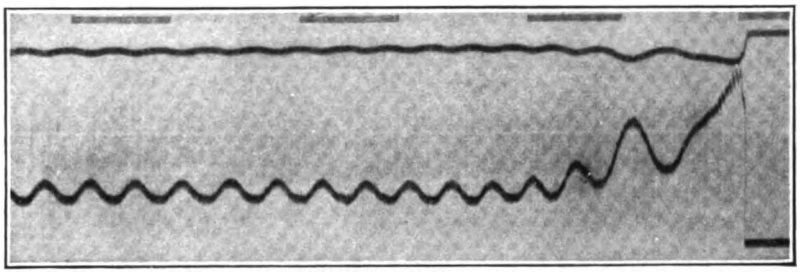

FIG. 13.—*Record as for Figs. 11 and 12, but at radiator temperature of 77° C*

The readings taken from these curves are given in Table 2

be seen by comparing the results for this car at 20° C with those at higher temperatures that the initial starting current does not decrease in the same proportion as the operating current, when the temperature is increased. This indicates that the initial current which the battery must supply depends more on the inertia of the system than on the temperature of the engine. The average operating current at 50° C is practically the same as the current at 77° C. A change is to be noted, however, in the amplitude of the fluctuations. At 20° C the amplitude is 36 amperes, at 50° C it is 28 amperes, and at 77° C it is 16 amperes. The significance of these changes will be brought out more clearly later on. On examination of these curves with reference to the time scale, it will be seen that the speed of the engine may be easily computed since there are two compressions for each revolution of the crank shaft. The values for the car E given in Table 2 have been computed from 11 oscillograms, which are not shown as illustrations. The power required initially to start the system in motion and the

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average operating power when the speed was constant show little change due to temperature above 35° C. It will be noticed in the table that slightly more power is required initially to start the system when the throttle is closed, but that the average operating power is less if the speed remains constant. The latter result may be due to effects produced by the greater compression in the cylinders when the throttle is open. As these observations were made in the summer it was not convenient to obtain temperatures below those given in the table. The power required to overcome the inertia of the system is somewhat less than the lowest figure given in column 10 of Table 2. The power required to overcome the compression is practically constant, irrespective of the temperature, provided the throttle opening is fixed. The power to overcome friction would probably increase rapidly at temperatures below those recorded here, as is indicated by Fig. 20. The increase in average operating power required for car E at the highest temperatures suggests increased friction. The engine had been run for a considerable period of time to increase the temperature. The oil and bearings had become heated and it is probable that the bearings were tighter at the higher temperatures because the engine had just been overhauled. Similar effects might also be produced by insufficient lubrication at these temperatures.

TABLE 2.—Instantaneous and Average Values of Current and Voltage Read from Figs. 11, 12, and 13 for the Car A and from Other Oscillograms Which Are Not Shown for the Car E

Car	Temperature of radiator water [a]	Engine speed	Throttle	Initial values		Average operating values		Amplitude of current	Power required	
				Current	Voltage	Current	Voltage		Initial	Average operating
	° C	r p m		Amp.	Volts	Amp.	Volts	Amp.	Watts	Watts
A	20	116.8	Closed	136	10.3	62	11.1	36	1400	688
A	50	162.6	do	127	11.1	38	11.8	28	1410	449
A	77	150.9	do	117	11.1	40	11.8	18	1300	472
E	25	89.0	Closed [b]	455	4.64	133	5.61	.....	2110	746
E	25	90.0	Open [b]	449	4.62	137	5.62	.....	2073	770
E	35	91.6	Closed	400	4.57	136	5.38	.....	1826	732
E	35	94.6	Open [b]	409	4.44	140	5.36	.....	1814	752
E	65	93.7	do	417	4.46	138	5.37	.....	1860	742
E	69	100.0	Closed	432	4.56	148	5.40	.....	1984	800
E	82	96.0	Open	417	4.42	154	5.38	.....	1842	829

[a] Atmospheric temperature for measurements on the car A, 25° C; for the car E, 28° C.
[b] Observations are the mean of two determinations.

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5. EFFECT OF COMPRESSION AND LUBRICATION

In order to clearly demonstrate that the fluctuations in the current curve are due to compression in the cylinders, a run was made in which one priming cup was removed. The results of this experiment on the car A are shown in Fig. 14. This illustration shows that there was onlv a slight compression in the cylinder that was open.

The power furnished by the battery is expended in several ways: First, there is a small part which is used up within the starter itself. This may be measured as in the case of any electric motor. A second part is expended in compressing the gas within the cylinders. A third part is expended in overcoming friction in the engine and

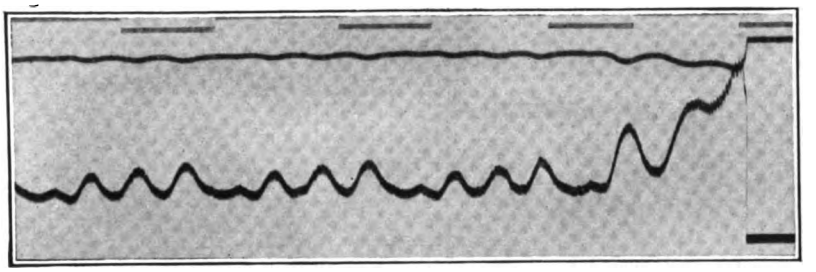

FIG. 14.—*Experiment to show the effect of compression on the record of current and voltage of the battery*

A pet cock was removed from one cylinder, car A. The compressive stroke in this cylinder was quicker than for the others, but the succeeding downward stroke occupied more time. As there was little compression in this cylinder no energy was returned to the system by compressed gas

varies with the speed and the lubrication. A fourth part is used for ignition on some cars. The amount of power which the battery furnishes may be obtained for any instant by multiplying the instantaneous values of current and voltage. If this product is also multiplied by an increment of time, we may obtain the total energy or work done by the battery during this instant of time.

It is apparent, therefore, that careful measurements of the electric power and time can furnish information as to the amount of work being done by the battery during any part of a cycle. One significant result of this may be seen in Fig. 14, where it will be observed that the engine speeds up in passing what would have been the compression stroke of the cylinder had the priming cup not been removed. As there was no compression in this cylinder, no energy could be returned to the system by compressed gas. The increased time required to reach the next maximum (Fig. 14) shows that the engine decreased in speed. This fact is determined

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by measuring the distance between the successive maximum values of the curve. A rolling effect results, which may be noticed by the difference in height of the successive maximum values. The watts delivered by the battery may be used as a sensitive and exact method of determining the instantaneous speed of the flywheel. The greater the inertia of the flywheel the smaller will be the fluctuations of the current and voltage curves.

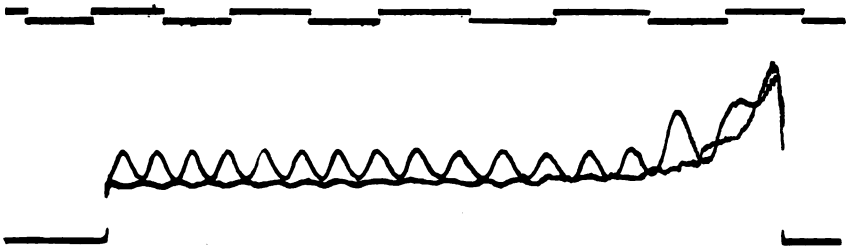

FIG. 15.—*Showing the effect of removing all spark plugs on the current curve, car A*

Two records have been superposed. Both were made with a radiator temperature of 20° C. For one the spark plugs were in place, for the other they were all removed

The curves of Figs. 11, 12, and 13 (car A) show a decreased amplitude at the higher temperatures. This suggests the conclusion that the amplitude for this type of engine depends on lubrication as well as compression. The effect due to friction is least at the top and bottom of the stroke and greatest when the crank arm and the connecting rod are at right angles. This result has been confirmed by experiments using one bank of the engine of car E.

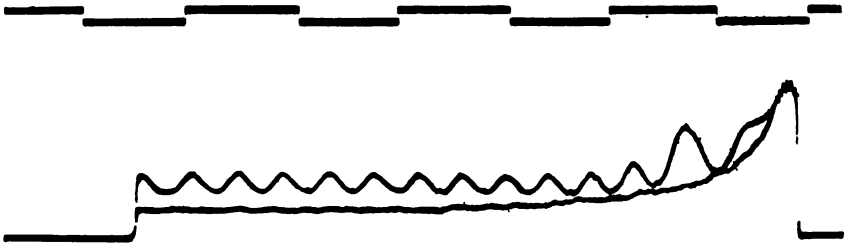

FIG. 16.—*Same as Fig. 15 but at radiator temperature of 47° C*

The curve for plugs out is lower than in Fig. 15 and higher than in Fig. 17.

It is possible to separate the effect of compression from that of friction and other losses by relieving the compression in all of the cylinders. This might best be done by removing the head of the engine, but in these experiments the compression was more conveniently relieved by removing the spark plugs. A series of comparative measurements, the results of which are shown in Figs.

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15, 16, and 17, were made on the car A at three different radiator temperatures, 20, 47, and 84° C respectively. These figures show that the curves obtained when the spark plugs were removed are devoid of the marked fluctuations of current observed when the spark plugs were in place. At each temperature a record of the current values was made with the spark plugs in, followed by an

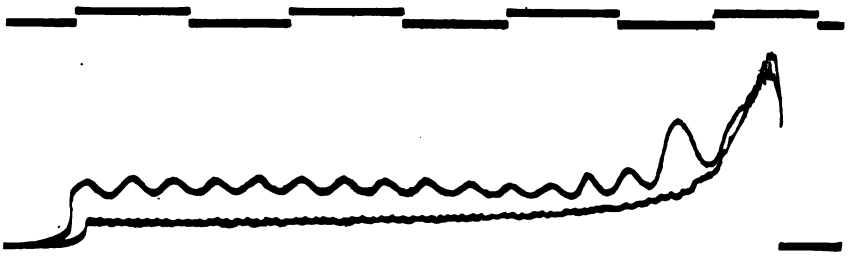

FIG. 17.—*Same as Figs. 15 and 16 but at radiator temperature of 84° C*

Values read from Figs. 15, 16, and 17 are used in Fig. 20

exactly similar measurement with the spark plugs removed. As there was no combustion in the cylinders the temperature of the radiator water is a fair indication of the temperature of the walls of the cylinders. The corresponding curves for each temperature have been superposed and traced together in Figs. 15, 16, and 17, care being taken to make them in perfect register, both as to the beginning of the curves and their position with respect to

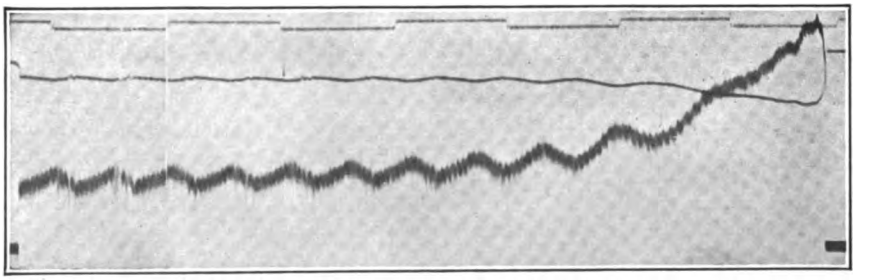

FIG. 18.—*Curves of current and voltage, car E, having all spark plugs removed from one bank of the engine*

Comparison of this with Fig. 9 shows the overlapping power strokes when all cylinders are in use

the horizontal axis. The results obtained at 20° C show that the curve without compression is practically tangent to the lower values of current when the spark plugs are in position. At the higher temperatures this curve falls somewhat below the minimum current values. Fig. 18 shows the results of an experiment on the car E with the spark plugs removed from the left bank. By

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comparing this with Fig. 9 the overlapping effect of the cylinders when all are in use may be seen. Fig. 19 shows the results of an experiment on the car E from which all spark plugs had been removed. The same high value of the initial current is to be noted in this figure as in Figs. 9 and 18. If Figs. 18 and 19 were superposed, the current curve of Fig. 19 would be tangent to the bottom of the current curve of Fig. 18.

The solid curves given in Fig. 20 show the relation between the temperature and viscosity of the oil used in car A. The values are plotted in terms of absolute units of viscosity and also in more familiar units, Saybolt seconds. The dotted curve gives the relation between the temperature and current in turning the engine after the inertia of the system had been overcome. These data were obtained from Figs. 15, 16, 17. The current is proportional

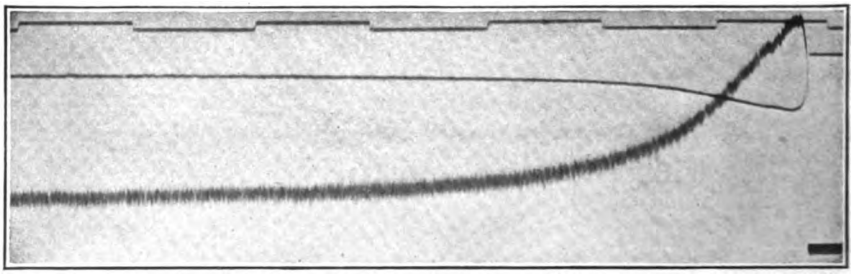

FIG. 19.—*Curve of current and voltage, car E, having all spark plugs removed*

The initial value of current in this figure is about 425 amperes, which is the same as for Fig. 9 when all the spark plugs were in place

to the torque. These curves are plotted to an arbitrary scale. The marked similarity suggests a decrease in friction with increasing temperature. These curves show a proportionality to exist between the viscosity of the oil and the power required of the battery to turn the engine.

A number of experiments were made on the car A at temperatures of 20, 50, and 80° C, to determine the power consumed by the starter system for various throttle openings. In Fig. 21 we have plotted the values read from the oscillograms for power and engine speed plotted against an arbitrary scale of throttle openings. We have also plotted the compression curve as measured by a gage for the same throttle openings as well as these could be determined without the use of fixed orifices in the carburetor.

It appears that if the amount of energy lost in the starter itself is known and deducted and the compression released by removing

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either the spark plugs or the head of the engine, the electrical power necessary to overcome friction may be calculated. In this way comparative measurements of the lubricating properties of different oils and friction losses in various parts of the machine might be made under running conditions.

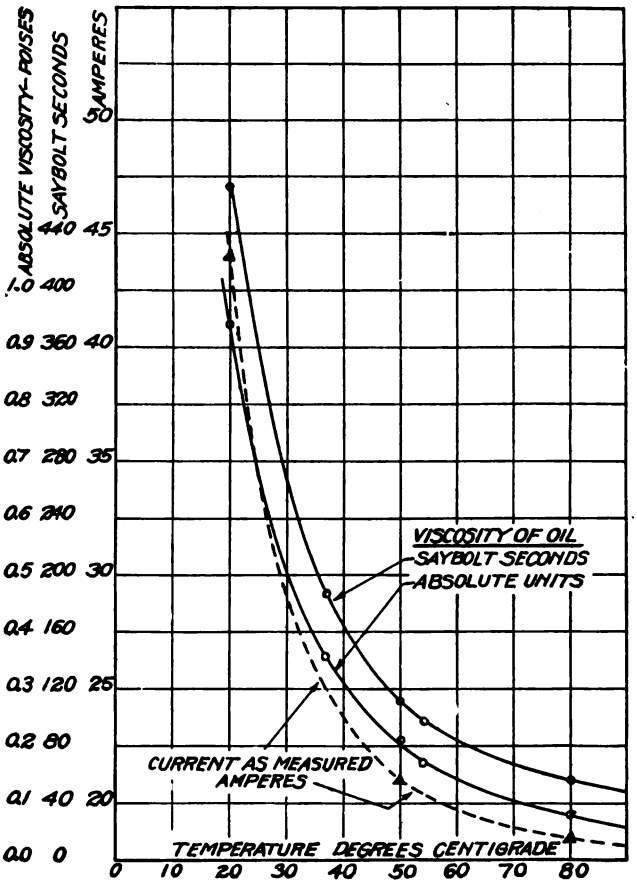

FIG. 20.—*Relation of electric current in starter circuit to viscosity of the oil used in the engine*

The currents measured in amperes as given by the curves of Figs. 15, 16, and 17, spark plugs removed, have been plotted against the temperature. The viscosity of the oil in absolute units has been plotted for comparison. The similarity indicates a proportionality between the viscosity and the current. The viscosity expressed in Saybolt seconds is given also

At this point it may be interesting to examine a curve which was obtained when one of the four cylinders of the car A was leaking. The exact cause of the leak is not known, but is supposed to have been due to a particle of carbon lodged in an exhaust valve. A curve showing the result is given in Fig. 22, in which it

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will be noted that every fourth maximum is considerably lower than the others.

Under this head there remains to be described one peculiar phenomenon which was observed in the experiment with the spark plugs out. It was observed in some cases where air was drawn directly into the cylinder, either by removing the spark plugs or taking out the priming cups, that greater demands were made on the battery in order to operate the starter after the spark

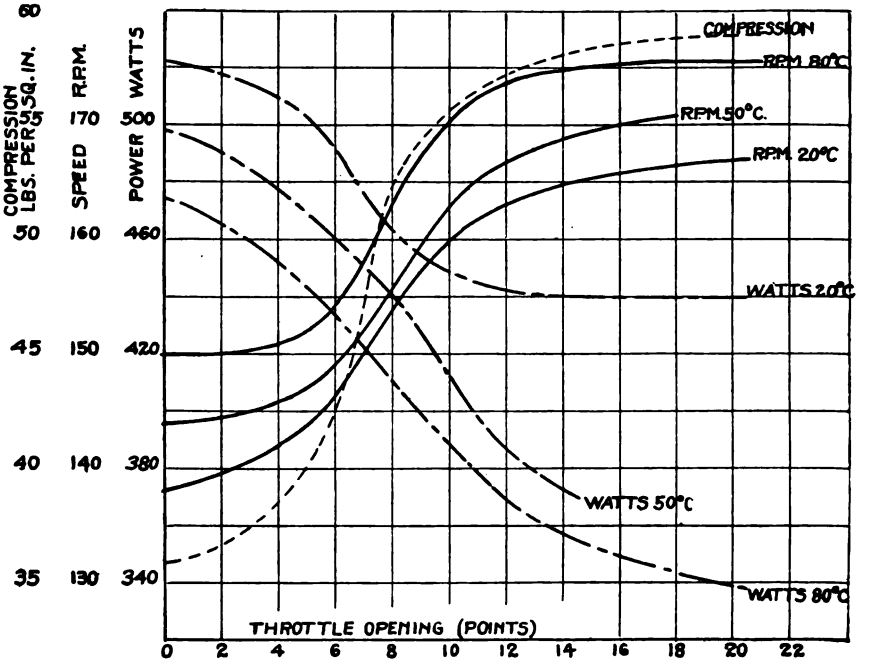

FIG. 21.—*Relation between throttle opening and electric power in watts delivered by the battery in cranking the engine of car A at 20, 50, and 80° C. The speed in r p m is also shown*

The dotted curve represents the compression in the cylinders measured by a gage for the same throttle openings as nearly as these could be repeated with the hand lever. The scale of throttle openings is arbitrary

plugs had been again replaced; that is to say, more work was required to turn the engine over. In Fig. 23 this effect is shown by the shape of the current curve. This record was made with a radiator temperature of 47° C, immediately after the spark plugs had been replaced. The curve shows the initial effort of the battery to have been exceeded by a second maximum which lasted for more than a half second, during which the engine did not turn. After gradually passing this maximum, the normal operation was restored. The same phenomenon is to be noticed in another curve, Fig. 24, which was made immediately after an experiment

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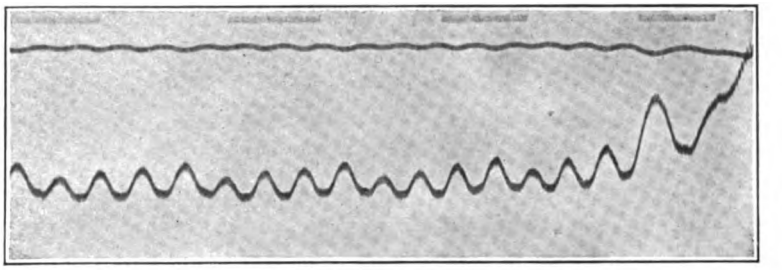

FIG. 22.—*Effect of a partly open valve in one cylinder, car A, ignition off*

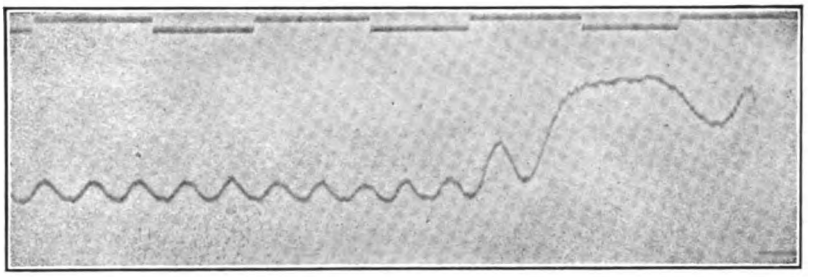

FIG. 23.—*Record showing a sticking of the engine after drawing air directly into the cylinders.*

The spark plugs had been removed for the experiments of Fig. 16 and replaced. Temperature, 47° Cs. The starter was unable to turn the engine for about 1 second

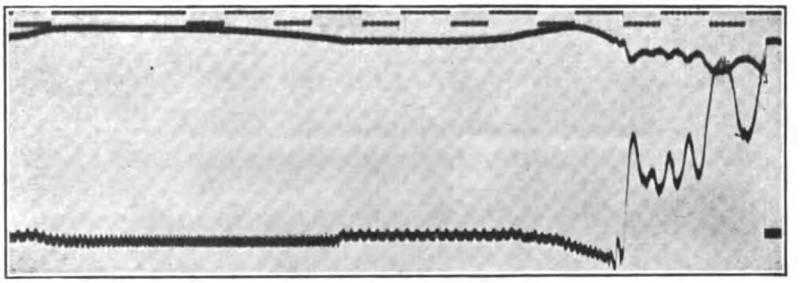

FIG. 24.—*Effect similar to that of Fig. 23, following the experiment of Fig. 14*

The pet cock had been replaced. The sticking effect at the first compression was repeated at the fifth

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in which a priming cup was removed from one cylinder. Here again it is observed that the first compression stroke, viz, that of the cylinder from which the priming cup had been removed and replaced, required as much power as was initially expended by the battery in starting the system in motion. The gas which was contained in the cylinder at this time was, of course, expelled by the succeeding stroke, but an excessive power consumption was again observed on the next compression in this cylinder as shown by the fourth maximum. The ignition switch was on during this experiment, and the engine began to operate on the next stroke, hence it is not possible to tell whether this effect persisted. The cause of this peculiar characteristic is not known.

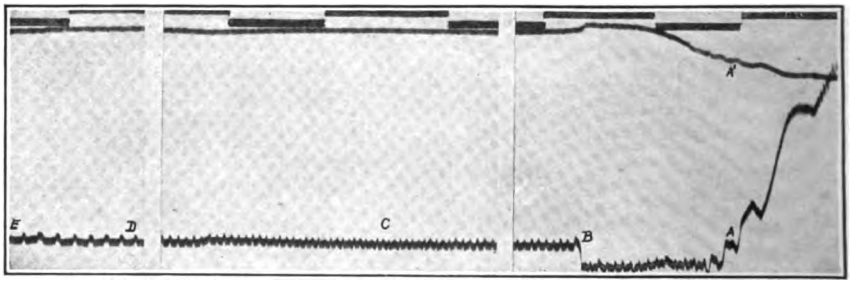

FIG. 25.—*The starting of the engine of car A and characteristics of the charging current*

A and *A'* represent the operation of the reverse-current relay; *B*, starter pedal released; *C*, engine speed increased; decreased at *D*; ignition cut off at *E*. Interruptions in charging current are due to contacts of ignition system

6. CHARACTERISTICS WITH THE ENGINE RUNNING

Experiments upon the car A similar to those shown in Figs. 11, 12, and 13 have been made with ignition on. One of the curves which has been thus obtained is shown in Fig. 25. In Fig. 25 the initial value of the current recorded is 136 amperes, which is exactly equal to that of Fig. 11. The initial value of the voltage, 10.3 volts, is also the same. The engine started on the second compression and the current values fell below the initial line, which shows that the battery was being charged. An immediate rise in the voltage curve to a point above open-circuit voltage shows this to be true. At the point *A* on the current curve and a corresponding point *A'* on the voltage curve is a short horizontal region where it is probable that the reverse-current relay operated. The charging voltage rises to 2.7 volts per cell, or 16.4 volts for the battery as a whole. This high charging voltage was due to the high rate at which the battery was

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being charged during the time that the starter switch was still closed. At the point marked *B* the starter switch was released. An immediate decrease in the charging voltage and the charging current was observed. The charging current decreased at regular intervals, the spacing of which varied with the speed of the engine. The engine was racing at *C* and slowed down at *D*. The ignition

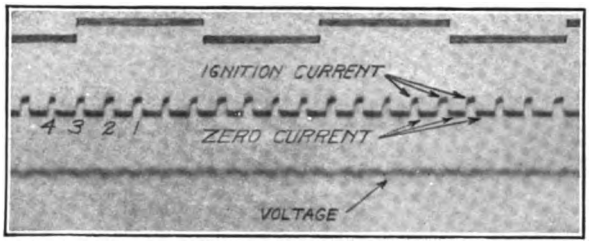

FIG. 26.—*Ignition current, car A, engine running slowly*

Note inequalities in time of contact

was cut off at *E*. It will be noted that these fluctuations in the charging current ceased at the time the ignition was cut off. Further study to determine the cause of these fluctuations showed that they occurred every time that the circuit was made through the primary coil of the ignition system. A magnification of these

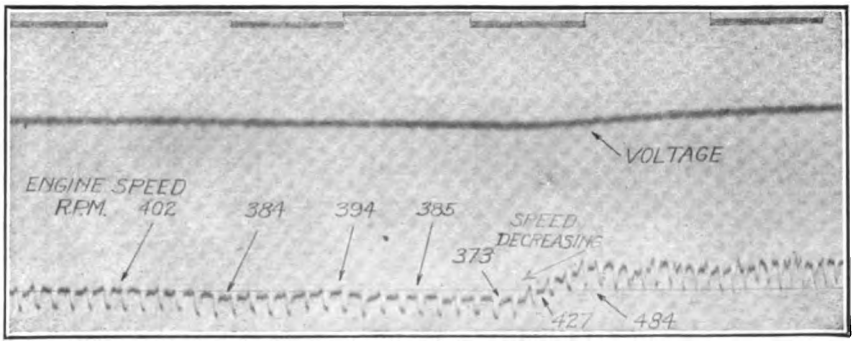

FIG. 27.—*Variation in charging current due to changing engine speed, car A*

The charging current changed to discharge current at 427 r p m of the crank shaft. The speed at any time can be calculated by counting the interruptions with respect to the time scale

current fluctuations is shown in Fig. 26. Here it will be seen that the duration of time that the current flowed through the primary coil was appreciably longer in the case of the contacts marked 1 and 2 than for those marked 3 and 4. This inequality in the action of the contact points is probably due to inequalities in the cam which operates the contact points, or possibly to wear in the

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distributor cam-shaft bearings. By comparing Fig. 26 with Fig. 10 it may be seen that the shape of the fluctuations in the current-charging curves are quite different although both had the same system for ignition. The difference is explained by the difference in the cams in the distributors.

Since the current passes through the primary of the ignition coil twice for each revolution of the crank shaft of the car A, it is possible to determine the exact speed of the engine by measuring the distance between any given number of these points in the charging curve with respect to the time scale. This is also possible for the car E, for which there are four contacts for each revolution. The exactness with which the speed of rotation may be measured by this method affords the possibility of determining small changes in the velocity of the flywheel system from one revolution to another, which can not be done by a tachometer.

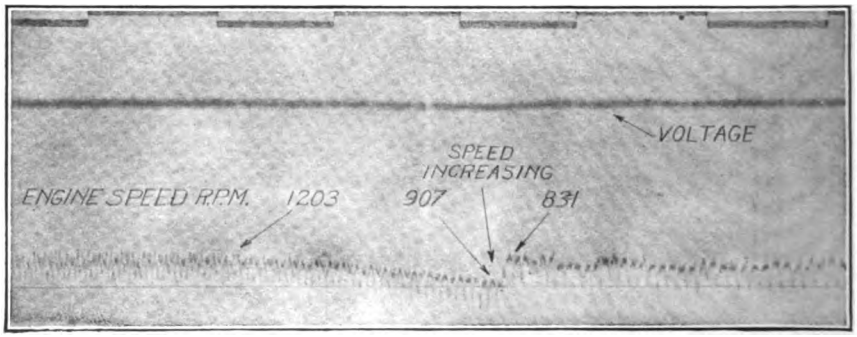

FIG. 28.—*Variations of charging rate at increasing speeds, car A*

The action of a limiting relay is shown at 907 r p m of the crank shaft

Figs. 27 and 28 show the characteristics of the charging current at varying speeds as observed on the car A. In Fig. 27 the speed was low, but in Fig. 28 it is much higher. The engine speed can be computed as described above. These curves are shown here to illustrate the applications of this method of measurement rather than to give quantitative measurements on this starter or system.

7. CONCLUSIONS AND SUMMARY

This investigation was begun to study the demands upon starting and lighting batteries in the operation of various types of automobiles. Illustrations of the oscillograms obtained for five of the cars measured have been included in this paper. These cars were loaned for these experiments by various individuals and

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by the Motor Transport Corps of the Army. Dr. H. C. Dickinson, of this Bureau, and Majors M. O. Boone and G. H. Totten, of the Motor Transport Corps, gave valuable suggestions and cooperation.

The results obtained enable us to determine the instantaneous values of current and voltage when the starting system is in operation. The interpretation of the records has also indicated the possibility of using this method for the study of lubrication and engine problems relating to torque, speed, friction, ignition, compression, distributor action, etc. A large number of oscillograms illustrating these applications have been given. These are intended to be suggestive rather than quantitative measurements of performance, since the observations were all made on cars in running condition, without fixed orifices on the carbureter or constant speed control.

WASHINGTON, October 6, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 187

A STUDY OF TEST METHODS FOR THE PURPOSE OF DEVELOPING STANDARD SPECIFICATIONS FOR PAPER BAGS FOR CEMENT AND LIME

BY

PAUL L. HOUSTON, Associate Physicist

Bureau of Standards

MARCH 21, 1921

PRICE, 5 CENTS

Sold only by the Superintendent of Documents, Government Printing Office
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A STUDY OF TEST METHODS FOR THE PURPOSE OF DEVELOPING STANDARD SPECIFICATIONS FOR PAPER BAGS FOR CEMENT AND LIME

By Paul L. Houston

ABSTRACT

This technologic paper contains information relating to the methods of testing and the apparatus employed in determining the quality of paper bags for lime and cement. A brief description is given of the ordinary tests performed on paper, such as weight in pounds of the standard size ream 25×40—500, bursting strength in points, tensile strength in kilograms, stretch in centimeters, folding endurance in number of double folds, percentage of fiber composition, percentage of ash, and percentage of rosin sizing. A special test is developed for giving numerically the stresses and strains that the paper of these bags undergo in service. This test is called a resiliency or endurance test. A service test is also given to determine the breaking strength of the paper when the bags are filled and dropped. A comparison is made of the results of this test with the results of the above resiliency test. Another service test is developed for determinining the strength of the adhesives used in these bags. The results of all these tests are tabulated, and from these data the very best quality bags are chosen. In the conclusions, special consideration is given this choice of best bags in determining the characteristics of a good quality bag, and specifications are drawn up accordingly.

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I. INTRODUCTION

1. PURPOSE OF TESTS

The purpose of these tests is to aid the paper-bag manufacturers to meet the requirements of the lime and cement manufacturers in obtaining a suitable paper bag in which to ship their product. For a number of years the paper-bag manufacturers have attempted to manufacture such an article to substitute for the cotton bag

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that had hitherto been used with great expense to the lime and cement industries because of the high price of cotton. The success of the paper-bag industry in this undertaking has been varied. A few of the consumers have given favorable reports on the use of their product, while others have claimed the bags fail in service. A number of consumers claim the glue gives way easily if the bags are held in storage any length of time. Since the bag manufacturers have been unable to derive any decided benefit from the above reports of the consumers, the present study was undertaken in order to develop specifications for these paper bags.

2. DISCUSSION OF MATERIALS AND APPARATUS USED

The materials used for this study were representative samples of paper bags obtained from a number of the leading bag manufacturers. The bags were given the following identification numbers: 14373, 14374, 14375, 14376, 14377, 14378, 14379, 14380, and 14381. The apparatus used was the Mullen paper tester, the paper scales, the 200 kg tensile-strength instrument, the 50 kg tensile-strength instrument, the folding endurance machine, microscope, glass slides, test tubes, extraction flasks, siphon cups, evaporating dishes and crucibles, all of which are used for ordinary physical and chemical tests. Descriptions and photographs of these instruments and apparatus may be obtained from circular of the Bureau of Standards No. 107, with the exception of the large 200 kg tensile-strength machine. The description and photograph of the tensile-strength apparatus in the above circular apply to the small 50 kg instrument. However, the 200 kg and the 50 kg instruments are practically identical in construction, the 200 kg machine being used for any material breaking at or under a load of 200 kg, the 50 kg machine for material breaking at or under a 50 kg load. In addition to the above paper-testing instruments a wooden platform with trapdoor and an ordinary foundry jolter were used for service tests. The wooden platform was built 3 feet high and allowed a bag filled with sand to be dropped through its trapdoor onto a solid wooden stand. The jolter consisted of a solid iron plate 3 inches thick which could be raised and lowered in rapid succession by means of air pressure. This machine, of course, was constructed with the necessary intake and outlet valves, air chamber, and piston to allow for the continuous jolting movement of the plate. For the purpose of developing a laboratory test that would give numerically the resiliency of bag

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paper or the stress and strain that the paper of a bag undergoes in service and by which the results of the above service tests could be checked, the large 200 kg tensile-strength machine was adopted with the following additional attachments: A brass cylinder was constructed and fastened to the upright standard in such a way that it would revolve. The revolution of this cylinder was controlled by a small wire over a pulley from the weighted pendulum so that, as the pendulum moved from its vertical position due to the increasing loads, the cylinder revolved accordingly. This cylinder held a paper chart, and a glass recording pen was used to record both load and stretch. The pen was controlled by a small wire extending from the lower-jawed arm over a pulley in such a way that, as the lower jaw descended and the paper stretched under the load, the pen ascended and described a curve which registered stretch. It can thus be seen that the revolution of the cylinder caused the registering of load on the chart, while the ascension of the pen caused the registering of stretch. The result was a curve on the chart giving both the increasing load and increasing stretch until the paper broke. In order to read this curve it was necessary to draw up the chart so that load and stretch could be read at a glance. The method used was as follows: An ordinary blank piece of writing paper was used for this purpose and placed in position on the cylinder. A point on the paper was chosen as the zero point for load and stretch. Then the zero line for stretch was found by using a piece of steel between the jaws and by applying the various loads. Steel was used for this purpose because the chart was to be drawn up for a 70 kg load as a maximum, and steel would not stretch at this load. By this method the zero line for stretch was drawn with the recording pen from the zero point, and each kilogram load as indicated by the scale was pointed off as the load increased. At each kilogram point a line was drawn parallel to the base or zero line for load. This base line was found by operating the machine with nothing between the jaws and was drawn with the recording pen from the zero point. Each centimeter that the lower jaw descended from its starting position or zero point represented stretch and was pointed off accordingly. At the different centimeter points lines were drawn parallel to the zero line for stretch. In this way the chart was plotted to give accurately the stretch at different loads for any paper breaking under a 70 kg load.

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3. SCOPE OF THE TESTS AND METHODS EMPLOYED

The object of these tests was, first, to discover all the physical and chemical properties of the paper of the bags; second, to check these results with the results of service tests on the bags themselves; third, to choose by the process of elimination the best bags of the series; and fourth, to develop specifications from this choice of best bags. For the ordinary physical and chemical tests the methods employed have been fully described in Circular of the Bureau of Standards No. 107, with the exception of the tensile strength and stretch tests. For these tests, the 200 kg instrument was used, and test samples were cut 2½ cm wide and long enough to allow 20 cm between jaws. In addition to the above tests a wet tensile test was conducted in the same manner as the dry tensile test described in the above Circular, with the following exceptions: The weight was removed from the pendulum of the 50 kg tensile-strength instrument, the machine was calibrated to find the corrections for the scale readings, and the paper was immersed for 10 minutes in water before testing. In both service tests bags were filled with 94 pounds of sand and then dropped or jolted until they broke open. In the jolter test the bags were jolted 1 inch high and 170 times per minute. An average of five tests was taken in each of these service tests to determine the strength of the bag. The method used for obtaining the resiliency of paper on the tensile-strength machine with its recording device was the following: In order to obtain curves or loops (as they should be called in this case) to show this resiliency or the endurance of paper under the repeated application of load, a load was taken in each case 10 per cent below the breaking point. This, of course, necessitated finding the tensile strength or breaking point for each grade of bag paper in both machine and cross direction. In order to do this an average of 10 tensile-strength tests was obtained. After obtaining in each case the desired load of 10 per cent below breaking point, each test sample was given this load, and then the load was released and the operation repeated until the test sample broke. All test samples for this work were cut 2½ cm wide and long enough to allow 20 cm of paper between jaws. Three test samples from each grade of paper, cut in both machine and cross direction, were used in order to obtain an average, and data were collected accordingly.

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II. EXPERIMENTAL WORK

1. PHYSICAL AND CHEMICAL TESTS

Each sample of bag paper was given the ordinary physical and chemical tests, such as weight in pounds of the standard size ream 25×40—500, bursting strength in points, tensile strength in kilograms, stretch in centimeters, folding endurance in number of double folds, fiber composition in per cent, per cent of ash, and per cent of rosin sizing. From these tests the ratio in per cent of bursting strength to weight in pounds of the standard size ream, 25×40—500, and the breaking length in yards were obtained as the best means for comparing papers of the same grade. An explanation and description of this ratio and breaking length may be procured from Circular of the Bureau of Standards No. 107. The results of these tests are compiled in Tables 1, 2, and 4. In addition to the above tests the paper and bags were given the following special physical tests: The paper was tested for tensile strength in grams when it was wet, and was tested also for resiliency or endurance. The numerical results are shown in Tables 2, 3, and 4 and are presented graphically in Figs. 1 to 54. The bags were tested for breaking strength by determining the number of times each bag could be dropped before failure, as shown in Table 5, and for strength of adhesive by determining the length of time each bag could be jolted before failure, as shown in Table 6. The purposes of these special tests were the following: The wet tensile-strength test was made in order to indicate what the strength of the paper of the bags would be when they became wet in shipment. The stress-strain or resiliency test was made in order to obtain a series of loops for each test sample, giving first the stretch at applied load, second the stretch when load was released, third the increase in stretch for every time load was applied over the previous time, and fourth the number of times the paper could stand the strain of this load before it broke. The purpose of the drop test was to reproduce service conditions, because a bag is often dropped while being handled. On the other hand, the jolter test not only tested the strength of the adhesive used in the bags but also somewhat duplicated the treatment that a bag undergoes while being jolted and jarred in shipment from manufacturer to consumer. The following data were collected accordingly from all the above tests, and from these data it was not hard, by process of elimination, to choose the five bag papers which showed the best bursting strength, tensile strength, stretch,

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folding endurance and resiliency, and which as bags showed the best breaking strength and possessed the strongest adhesive. At a glance, bag papers numbered 14373, 14374, 14378, 14379, and 14381 seem to be the strongest. In comparing the following results of the resiliency test with those results of the drop test, it will be noted that the above papers proved strongest in both cases. This proves that the resiliency test is a very good test for showing what the paper of a bag will do in actual service, and since it is strictly a laboratory test, it is highly to be recommended.

2. TABLES OF RESULTS

TABLE 1.—Results of Ordinary Physical and Chemical Tests on Cement and Lime Bag Paper

Bag-paper identification numbers	Weight of the standard size ream 25×40–500	Bursting strength in points	Ratio of bursting strength to weight of the standard size ream 25×40–500	Percentage fiber composition	Ash	Rosin sizing
	Pounds		Per cent		Per cent	Per cent
14373	150	134.0	89.3	100 jute and manila	4.10	3.40
14374	119	100.0	84.0	.....do	3.60	3.80
14375	143	108.0	75.6	95 jute and manila, 5 chemical wood	4.70	4.90
14376	127	96.0	75.6	100 chemical wood	3.30	3.30
14377	115	78.0	67.8	100 jute and manila	4.60	4.10
14378	176	132.0	75.0	50 jute and manila, 50 chemical wood	2.30	4.40
14379	142	143.0	107.0	75 jute and manila, 25 chemical wood	2.00	3.90
14380	172	159.0	92.4	100 chemical wood	1.27	.97
14381	181	155.0	85.7	.....do	.97	1.35

TABLE 2.—Results of Tensile-Strength Tests and Stress-Strain or Resiliency Tests on Cement and Lime Bag Paper

Bag-paper identification numbers	Weight of the standard size ream 25×40—500	Load at breaking point		Stretch at breaking point		Breaking length		Load 10 per cent below breaking point		Number of times load is applied	
		Machine direction	Cross direction	Machine direction	Cross direction	Machine direction	Cross direction	Machine direction	Cross direction	Machine direction	Cross direction
	Pounds	kg	kg	cm	cm	Yards	Yards	kg	kg		
14373	150	35.0	17.5	0.57	1.57	7130	3565	31.50	15.75	5.66	8.66
14374	119	28.0	16.5	.64	1.60	7160	4240	25.20	14.85	21.66	12.33
14375	143	27.0	11.0	.49	1.25	5770	2350	24.30	9.90	30.30	19.33
14376	127	26.5	13.0	.60	1.43	6380	3140	23.85	11.70	26.00	8.66
14377	115	21.5	8.8	.30	.70	5710	2340	19.35	7.92	11.00	11.66
14378	176	46.5	18.0	.54	1.34	5100	3120	41.85	16.20	49.66	4.66
14379	142	37.5	17.5	.64	1.06	8060	3770	33.75	15.75	12.00	9.66
14380	172	43.4	16.4	.45	1.12	7730	2910	39.06	14.76	1.00	2.00
14381	181	42.3	18.4	.50	1.14	7160	3110	38.07	16.56	59.00	9.66

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TABLE 3.—Results of Stress-Strain or Resiliency Tests on Cement and Lime Bag Paper, Showing Different Stretches

	Loop 1	Loop 3	Loop 5	Loop 10	Loop 20	Loop 30	Loop 40	Loop 60
Bag identification No. 14373:								
Stretch at applied load—								
Machine direction.....per cent..	1.68	2.01	2.30	.....	.....	.....	.....	.....
Cross direction.....do....	5.66	5.77	.....	6.10	.....	.....	.....	.....
Stretch when load is released—								
Machine direction.....centimeters..	.25	.36	.40	.....	.....	.....	.....	.....
Cross direction.....do....	.90	.91	1.03	1.05	.....	.....	.....	.....
Bag identification No. 14374:								
Stretch at applied load—								
Machine direction.....per cent..	2.60	.....	.....	2.65	2.75	.....	2.95	3.15
Cross direction.....do....	6.28	6.96	7.56	7.80	.....	.....	.....	.....
Stretch when load is released—								
Machine direction.....centimeters..	.30	.....	.42	.47	.51	.....	.54	.56
Cross direction.....do....	.94	1.27	1.34	1.44	.....	.....	.....	.....
Bag identification No. 14375:								
Stretch at applied load—								
Machine direction.....per cent..	1.18	.....	1.56	1.90	2.22	2.60	.....	.....
Cross direction.....do....	3.85	5.13	.....	.....	5.40	.....	5.60	.....
Stretch when load is released—								
Machine direction.....centimeters..	.27	.....	.36	.44	.50	.....	.....	.....
Cross direction.....do....	.53	.86	.....	.....	.92	.....	1.00	.....
Bag identification No. 14376:								
Stretch at applied load—								
Machine direction.....per cent..	1.75	.....	2.16	2.51	2.75	.....	.....	.....
Cross direction.....do....	5.28	5.90	6.50	.....	6.60	.....	.....	.....
Stretch when load is released—								
Machine direction.....centimeters..	.31	.38	.44	.51	.60	.....	.....	.....
Cross direction.....do....	.80	.99	.....	1.02	.....	.....	.....	.....
Bag identification No. 14377:								
Stretch at applied load—								
Machine direction.....per cent..	1.16	.....	1.65	2.00	.....	.....	.....	.....
Cross direction.....do....	2.90	3.75	.....	4.05	4.30	.....	.....	.....
Stretch when load is released—								
Machine direction.....centimeters..	.23	.....	.36	.43	.....	.....	.....	.....
Cross direction.....do....	.40	.67	.....	.70	.75	.....	.....	.....
Bag identification No. 14378:								
Stretch at applied load—								
Machine direction.....per cent..	1.68	.....	.....	1.90	.....	.....	.....	.....
Cross direction.....do....	5.46	6.38	7.12	.....	.....	.....	.....	.....
Stretch when load is released—								
Machine direction.....centimeters..	.26	.....	.32	.40	.....	.....	.....	.....
Cross direction.....do....	.82	1.06	1.20	.....	.....	.....	.....	.....
Bag identification No. 14379:								
Stretch at applied load—								
Machine direction.....per cent..	2.31	.....	2.55	2.85	3.10	.....	.....	.....
Cross direction.....do....	4.93	5.28	5.45	5.60	6.00	.....	.....	.....
Stretch when load is released—								
Machine direction.....centimeters..	.38	.50	.58	.64	.70	.....	.....	.....
Cross direction.....do....	.80	.95	1.00	1.05	.....	.....	.....	.....
Bag identification No. 14380:								
Stretch at applied load—								
Machine direction.....per cent..	2.10	.....	.....	.....	.....	.....	.....	.....
Cross direction.....do....	3.13	3.60	.....	.....	.....	.....	.....	.....
Stretch when load is released—								
Machine direction.....centimeters..	.....	.....	.....	.....	.....	.....	.....	.....
Cross direction.....do....	.50	.78	.....	.....	.....	.....	.....	.....
Bag identification No. 14381:								
Stretch at applied load—								
Machine direction.....per cent..	1.53	.....	1.83	2.10	2.33	.....	.....	.....
Cross direction.....do....	3.13	3.90	4.50	4.95	.....	.....	.....	.....
Stretch when load is released—								
Machine direction.....centimeters..	.32	.....	.42	.49	.54	.....	.....	.....
Cross direction.....do....	.44	.56	.66	.72	.....	.....	.....	.....

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TABLE 4.—Results of Wet Tensile Tests and Folding Endurance Tests on Cement and Lime Bag Paper

Bag-paper identification numbers	Wet tensile strength		Breaking length		Folding endurance in number of double folds	
	Machine direction	Cross direction	Machine direction	Cross direction	Machine direction	Cross direction
	kg	kg	Yards	Yards		
14373	1.38	1.40	476	482	10 000+	10 000+
14374	2.36	1.71	1026	746	10 000+	10 000+
14375	1.43	1.29	519	468	10 000+	1 998
14376	2.05	1.68	836	685	6 220	4 260
14377	1.17	.92	525	418	10 000+	5 011
14378	3.15	1.35	926	396	10 000+	10 000+
14379	1.31	.71	349	262	10 000+	10 000+
14380	.80	.30	243	90	7 244	10 000+
14381	1.26	.69	362	199	10 000+	10 000+

TABLE 5.—Results of Drop Service Tests on Cement and Lime Bag Paper

Bag-paper identification numbers	Position of bag when dropped	Number of drops before failure	Extent of failure
14373	On side and seam up	2	Paper failed at seam.
14374	do	3	Do.
14375	do	1	Paper and glue failed at seam.
14376	do	1	Paper failed across edge, and glue failed at end.
14377	do	1	Paper and glue failed at seam, and paper failed across bottom side.
14378	do	4	Paper failed at seam.
14379	do	4	Do.
14380	do	1	Paper failed across end, from side to side, and glue failed at end.
14381	do	3	Glue failed at seam and end.

TABLE 6.—Results of Jolter Service Tests on Cement and Lime Bag Paper

Bag-paper identification numbers	Position of bag when jolted	Minutes jolted before failure	Extent of failure
14373	On side and seam up	24	Glue failed at end.
14374	do	60	No failure.
14375	do	1	Glue failed at end.
14376	do	18	Do.
14377	do	6	Do.
14378	do	40	Do.
14379	do	30	Do.
14380	do	2	Do.
14381	do	3	Do.

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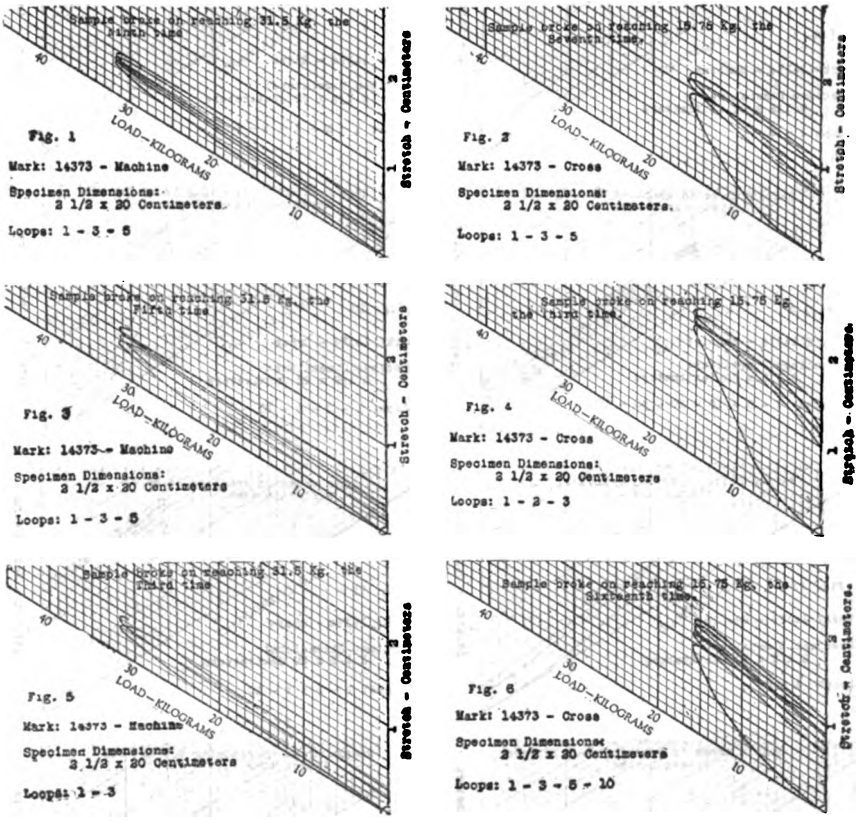
3. DIAGRAMS ILLUSTRATING RESULTS



FIGS. 1 to 6.—*Diagrams showing resiliency or endurance of cement and lime bag paper in the machine and cross directions*

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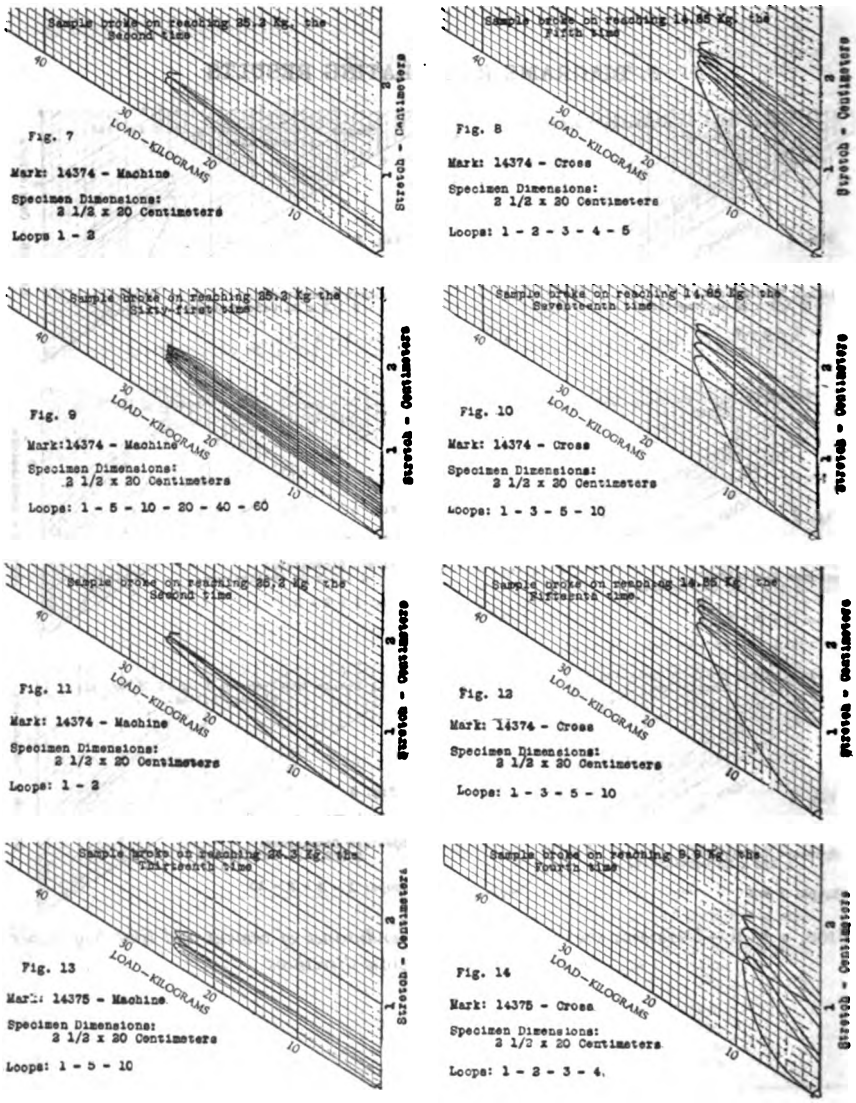
Sample broke on reaching 25.2 kg. the second time
Fig. 7
Mark: 14374 - Machine
Specimen Dimensions: 2 1/2 x 20 Centimeters
Loops 1 - 2
Sample broke on reaching 14.85 kg. the first time
Fig. 8
Mark: 14374 - Cross
Specimen Dimensions: 2 1/2 x 20 Centimeters
Loops: 1 - 2 - 3 - 4 - 5
Sample broke on reaching 25.2 kg the sixty-first time
Fig. 9
Mark:14374 - Machine
Specimen Dimensions: 2 1/2 x 20 Centimeters
Loops: 1 - 5 - 10 - 20 - 40 - 60
Sample broke on reaching 14.85 kg. the seventeenth time
Fig. 10
Mark: 14374 - Cross
Specimen Dimensions: 2 1/2 x 20 Centimeters
Loops: 1 - 3 - 5 - 10
Sample broke on reaching 25.2 kg the second time
Fig. 11
Mark: 14374 - Machine
Specimen Dimensions: 2 1/2 x 20 Centimeters
Loops: 1 - 2
Sample broke on reaching 14.85 kg. the fifteenth time
Fig. 12
Mark: 14374 - Cross
Specimen Dimensions: 2 1/2 x 20 Centimeters
Loops: 1 - 3 - 5 - 10
Sample broke on reaching 26.3 kg. the thirteenth time
Fig. 13
Mark: 14375 - Machine
Specimen Dimensions: 2 1/2 x 20 Centimeters
Loops: 1 - 5 - 10
Sample broke on reaching 9.9 kg. the fourth time
Fig. 14
Mark: 14375 - Cross
Specimen Dimensions: 2 1/2 x 20 Centimeters
Loops: 1 - 2 - 3 - 4
LOAD - KILOGRAMS
Stretch - Centimeters

FIGS. 7 to 14.—*Diagrams showing resiliency or endurance of cement and lime bag paper in the machine and cross directions*

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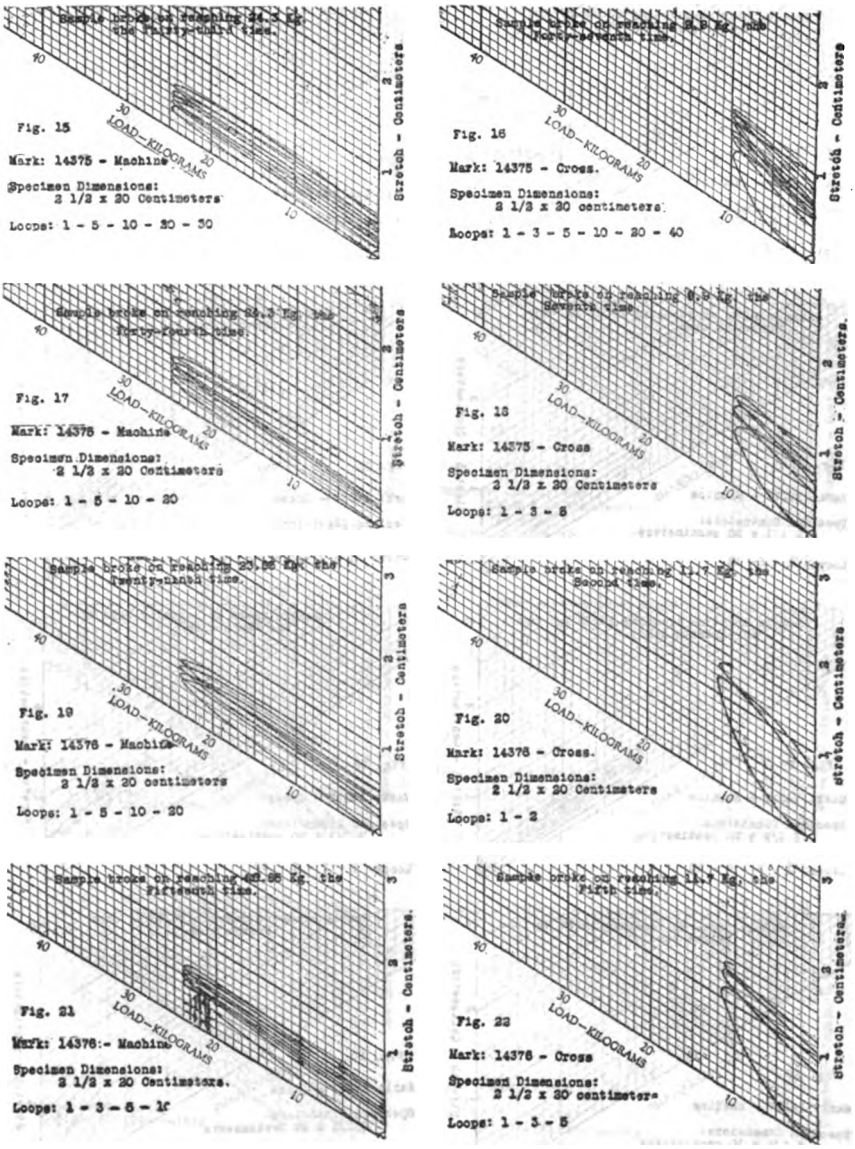
Fig. 15
Mark: 14375 - Machine
Specimen Dimensions:
2 1/2 x 20 Centimeters
Loops: 1 - 5 - 10 - 20 - 30
Fig. 16
Mark: 14375 - Cross.
Specimen Dimensions:
2 1/2 x 20 centimeters.
Loops: 1 - 3 - 5 - 10 - 20 - 40
Fig. 17
Mark: 14376 - Machine
Specimen Dimensions:
2 1/2 x 20 Centimeters
Loops: 1 - 5 - 10 - 20
Fig. 18
Mark: 14375 - Cross
Specimen Dimensions:
2 1/2 x 20 Centimeters
Loops: 1 - 3 - 5
Fig. 19
Mark: 14376 - Machine
Specimen Dimensions:
2 1/2 x 20 centimeters
Loops: 1 - 5 - 10 - 20
Fig. 20
Mark: 14376 - Cross.
Specimen Dimensions:
2 1/2 x 20 Centimeters
Loops: 1 - 2
Fig. 21
Mark: 14376 - Machine
Specimen Dimensions:
2 1/2 x 20 Centimeters.
Loops: 1 - 3 - 5 - 10
Fig. 22
Mark: 14376 - Cross
Specimen Dimensions:
2 1/2 x 20 centimeters
Loops: 1 - 3 - 5
LOAD - KILOGRAMS
Stretch - Centimeters

FIGS. 15 to 22.—*Diagrams showing resiliency or endurance of cement and lime bag paper in the machine and cross directions*

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III. CONCLUSIONS

Conclusions to be drawn from the above results of tests, taking into special consideration the various tests on the chosen five best bags, are that a satisfactory cement or lime bag paper should have a very good bursting strength and a high tensile strength and stretch in both directions. It must also have a high endurance or resiliency, which is determined by the simple drop service test or by the better stress-strain laboratory test. A suitable bag paper must also show a good tensile strength when wet and must have a high folding endurance. As the above data show, it is not necessary to manufacture a bag out of very heavy paper in order to obtain a bag of the very best quality. For instance, bag paper numbered 14374 weighs only 119 pounds to the standard size ream 25×40—500, and yet is one of the five best papers tested. The fiber composition of a good bag paper should be not less than 50 per cent strong manila and jute and the remainder chemical wood. The ash should not be over 3 per cent and the rosin sizing should be at least 3.5 per cent.

IV. SPECIFICATION

In writing up the specification for cement and lime paper bags it does not seem wise to recommend any specific weight of paper, since this study shows that weight alone should not be considered in choosing the best-quality bags. It is for the consumer to decide what weight bag he wishes to use and for the manufacturer to decide what weight bag paper he wishes to manufacture. However, it is thought that both manufacturer and consumer would prefer a light-weight paper for reasons of economy. For these reasons weight is not given in the following specification, but in all other respects it is thought that this specification will meet all requirements and can be adhered to by the manufacurers without any difficulty, and that bags made of paper that conforms to this specification will prove satisfactory to the cement and lime manufacturers.

SPECIFICATIONS FOR PAPER BAGS FOR CEMENT AND LIME

Bursting strength: Not less than 100 points.

Ratio bursting strength to weight in pounds of the standard-size ream, 25×40—500: Not less than 75 per cent.

Fiber composition: Not less than 50 per cent manila and jute; the remainder as chemical wood.

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Ash: Not over 3 per cent.

Rosin: Not less than 3.5 per cent.

Tensile strength (test sample 2½ cm wide and 20 cm between jaws): Not less than 28 kg (machine direction), 16 kg (cross direction).

Breaking length: Not less than 7000 yards (machine direction), 3500 yards (cross direction).

Wet tensile strength (test sample 15 mm wide and 90 mm between jaws): Not less than 2000 g (machine direction), 1300 g (cross direction).

Wet breaking length: Not less than 800 yards (machine direction), 400 yards (cross direction).

Folding endurance (test sample 15 mm wide and 90 mm between jaws): Not less than 10 000 double folds (machine direction), 10 000 double folds (cross direction).

Resiliency strength (test sample 2½ cm wide and 20 cm between jaws): Not less than 20 times (machine direction), 9 times (cross direction).

Breaking strength of bags: Not less than 3 times.

Adhesive strength: Not less than 30 minutes.

WASHINGTON, September 20, 1920.

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No. 188

SOME PROPERTIES OF WHITE METAL BEARING ALLOYS AT ELEVATED TEMPERATURES

BY

JOHN R. FREEMAN, Jr., Associate Physicist
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Bureau of Standards

APRIL 5, 1921

PRICE, 5 CENTS
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SOME PROPERTIES OF WHITE METAL BEARING ALLOYS AT ELEVATED TEMPERATURES

By John R. Freeman, Jr., and R. W. Woodward

ABSTRACT

An apparatus is described for determining the yield point and ultimate strength of white metal bearing alloys at temperatures up to 100° C. A new design of heating apparatus is described for determining the Brinell hardness of metals at temperatures up to 100° C.

The results of compression tests and Brinell hardness tests at temperatures up to 100° C are given for five typical white metal bearing alloys, including three tin-base alloys, one lead-base alloy, and one intermediate alloy, which show that the tin-base alloys maintain their properties better at elevated temperatures than the lead-containing alloys.

Results of tests are given which indicate that up to 5 per cent of lead in a high-grade babbitt does not affect the yield point or ultimate strength at 25 or 75° C.

The yield point of tin-base alloys is not affected by heating for six weeks at about 100° C, but the yield point is lowered in the lead-base alloy by heating for two weeks at about 100° C.

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I. INTRODUCTION

The mechanical properties of white metal bearing alloys have been the subject of several investigations with the particular object of establishing the relations between the properties obtained in laboratory tests and the ultimate test of service. Practically all of these tests reported in the literature on white metal bearing alloys were conducted at room temperature, the Brinell hardness alone[1] having been determined for a few alloys under other conditions of temperature. While we therefore have considerable knowledge of the mechanical properties of these alloys at ordinary temperatures, our knowledge of their properties at elevated temperatures is very limited.

The importance of knowing the properties of bearing alloys at elevated temperatures is readily appreciated when one considers that the oil temperature in the crank case of an automobile engine may often reach 60° C, and that bearing temperatures of 100° C and higher have been measured in similar engines.

It is the purpose of this paper to present the design of new apparatus and the results of tests to determine the mechanical properties in compression and the Brinell hardness of some representative white metal bearing alloys at elevated temperatures.

The nonferrous metals division of the Society of Automotive Engineers is proposing as standard white metal bearing alloys the four compositions given in Table 1.

TABLE 1.—Specifications for White Metal Bearing Alloys Proposed by the Society of Automotive Engineers

Components	S. A. E. No. 10	S. A. E. No. 11	S. A. E. No. 12	S. A. E. No. 13
	Per cent	Per cent	Per cent	Per cent
Tin	90–92	86–89	Remainder	4.50–5.50
Copper	4–5	5–6.50	2.25–3.75	<0.50
Antimony	4–5	6–7.50	9.50–11.50	9.25–10.75
Lead	<0.35	<0.35	24–26	84–86

These four alloys and A. S. T. M. alloy No. 2[2] were selected as of representative composition suitable for this investigation. It will be noticed that A. S. T. M. alloy No. 2 (Table 2) is very similar

[1] Jesse L. Jones, Babbitt and babbitted bearings, A. I. M. M. E. Trans., **60**, p. 458; 1919.

[2] "Tentative specifications for white metal bearing alloys," Proc. A. S. T. M., **19**, pt. 1, p. 469.

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to the S. A. E. alloy No. 11. The properties of these two alloys are compared here, as the A. S. T. M. alloy was considered by the S. A. E. committee as a little hard.

The results of chemical analyses[3] of the five alloys studied are given in Table 2.

TABLE 2.—Percentage Composition of Alloys Studied

Components	Bearing metal No. 1: S. A. E. No. 10; A. S. T. M. No. 1	Bearing metal No. 2: A. S. T. M. No. 2	Bearing metal No. 3: S. A. E. No. 11	Bearing metal No. 4: S. A. E. No. 12	Bearing metal No. 5; S. A. E. No. 13; A. S. T. M. No. 9
	Per cent	Per cent	Per cent	Per cent	Per cent
Copper	4.96	3.51	5.65	2.90	.....
Antimony	4.52	7.57	6.90	10.50	10.03
Tin	Remainder	Remainder	Remainder	Remainder	Remainder
Lead	None	None	0.09	25.05	84.95
Iron	<0.05	<0.05	< .05	< .05	< .05

II. PREPARATION OF ALLOYS

1. METALS USED

Pure Banka tin and the best grade of "Star" antimony were used. Neither these metals nor the copper were analyzed, as the freedom of the alloys from impurities is proof of the purity of the metals used.

Analysis of the commercially pure lead used showed the following: Lead, 99.94 per cent; copper, 0.03 per cent; antimony, <0.03 per cent.

2. ALLOYING PROCEDURE

Alloys Nos. 1, 2, 3, and 4 were prepared by first melting the tin in a plumbago crucible in a gas furnace and then adding the requisite amounts of a 50 per cent Sn.-50 per cent Cu hardener and metallic antimony. After addition of the alloying elements the temperature of the bath was carried up to the melting point of antimony and stirred to insure a homogeneous alloy. The temperature of the melt was then allowed to drop to about 500° C with continual stirring of the metal, which was then poured into a cast-iron mold. The surface of the bath was always kept covered with charcoal to prevent excessive oxidation. The temperature was measured by means of a specially calibrated chromel-alumel thermocouple connected to a portable potentiometer.

[3] J. A. Scherrer, of the Bureau of Standards, made all chemical analyses reported in this paper.

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The lead-base alloy No. 5 was similarly prepared, but in this case the lead was first melted and then the metallic tin and antimony were added.

The alloys were made up to meet the mean composition of the specifications, and the resultant compositions given indicate how close a desired composition may be obtained with the careful laboratory methods used.

III. PREPARATION OF TEST SPECIMENS

The compression test specimens used were small cylinders 1½ inches long by about ½ inch diameter (1.5 by 0.514 inch), this ratio of length to diameter being within the limits recommended by the A. S. T. M., and the cross section for these experiments being easily tested in a 10 000-pound testing machine. These specimens were turned in a lathe, with a hollow mill, from castings 2 inches long by ¾ inch diameter which were made by pouring the metal from the desired temperature into a split steel mold of the above dimensions.

The samples for Brinell testing were similar to those used by Lynch,[4] the metal being poured into an open steel mold 2 inches in diameter by ⅝ inch deep, but in this case the mold was not previously heated before pouring, always being at room temperature when the metal was first poured. Before making the impressions, the faces of the casting were turned off and the test then made on the bottom face. Three impressions were made on each casting at equidistant points on a circle one-half the radial distance from the center. The average of these three readings was taken as the Brinell hardness under the given conditions.

IV. APPARATUS USED FOR TESTING

1. COMPRESSION TESTS

The cylinders were compressed in a standard Reihle 10 000-pound testing machine.

The deformation per unit load was measured by a specially designed compressometer. A copy of a photograph of this instrument mounted on a specimen is shown in Fig. 1. The frame and uprights are made of aluminum. They are held to the specimen by three small steel screws, having conical points, set radially in the same plane, and spaced equidistantly around

[4] T. D. Lynch, "Study of bearing metals and methods of testing," A. S. T. M., 18, p. 699; 1913.

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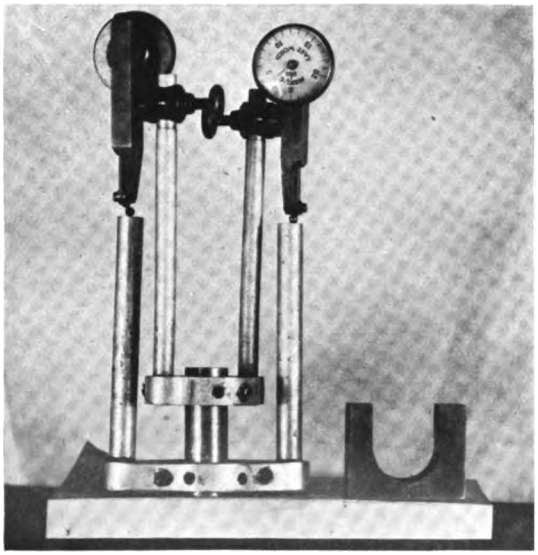

FIG. 1.—*Compressometer mounted on specimen*

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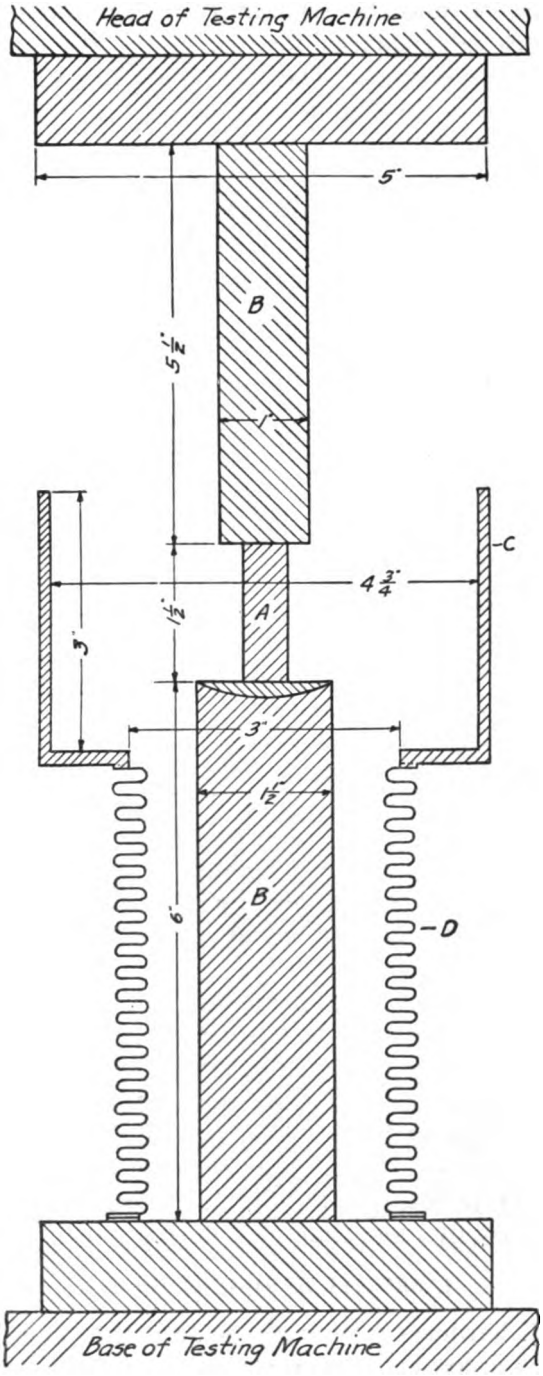

FIG. 2.—*Assembly drawing of a section of heating bath, showing original dimensions*

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the specimen, as may be seen in the photograph. The small U-shaped block is a gage used for spacing the frames at the proper distance on the specimen. The gage length or the distance between the planes of the screws is 1 inch.

The "Last Word" dials used read to approximately thousandths of an inch (one division on dial = 0.00087 in.), and ten thousandths are readily estimated.

The assembly and important dimensions of the bath used for heating the specimen during tests are shown in Fig. 2, which is a section through the center. The specimen *A* is compressed between the steel posts *B B*. During test the specimen with compressometer attached is immersed in a heated liquid (glycerin was found very satisfactory) held in the container *C*. *D* is a Silphon diaphragm. This collapses like an accordion, permitting the top of the container *C* to drop below the level of the base of the specimen. This is a particularly convenient method for lowering the bath to place a specimen in position for testing, especially as it eliminates the need for any packed joints. A photograph of the entire apparatus with a specimen in position for testing is shown in Fig. 4.

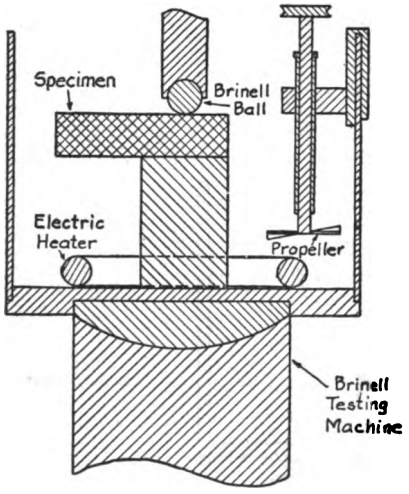

FIG. 3.—*Apparatus for Brinell hardness testing at elevated temperatures*

The bath is heated with a small size Hot-Point electric heater immersed in the glycerin. The glycerin was forced in a continuous stream over the heater by a small electric motor-driven propeller. This continuous stirring of the glycerin and the ready control of the heating current with a variable resistance provided excellent control of the temperature of the specimen during the test. In all cases the temperature of the bath and consequently the specimen did not vary by more than 2° C during a test.

2. BRINELL HARDNESS TESTS

The Brinell hardness tests were made with a standard Brinell machine using a 500 kg load on a 10 mm ball applied for 30 seconds. For the elevated temperature tests the apparatus shown in Fig. 3 was used. It is simply a suitable container for the heat-

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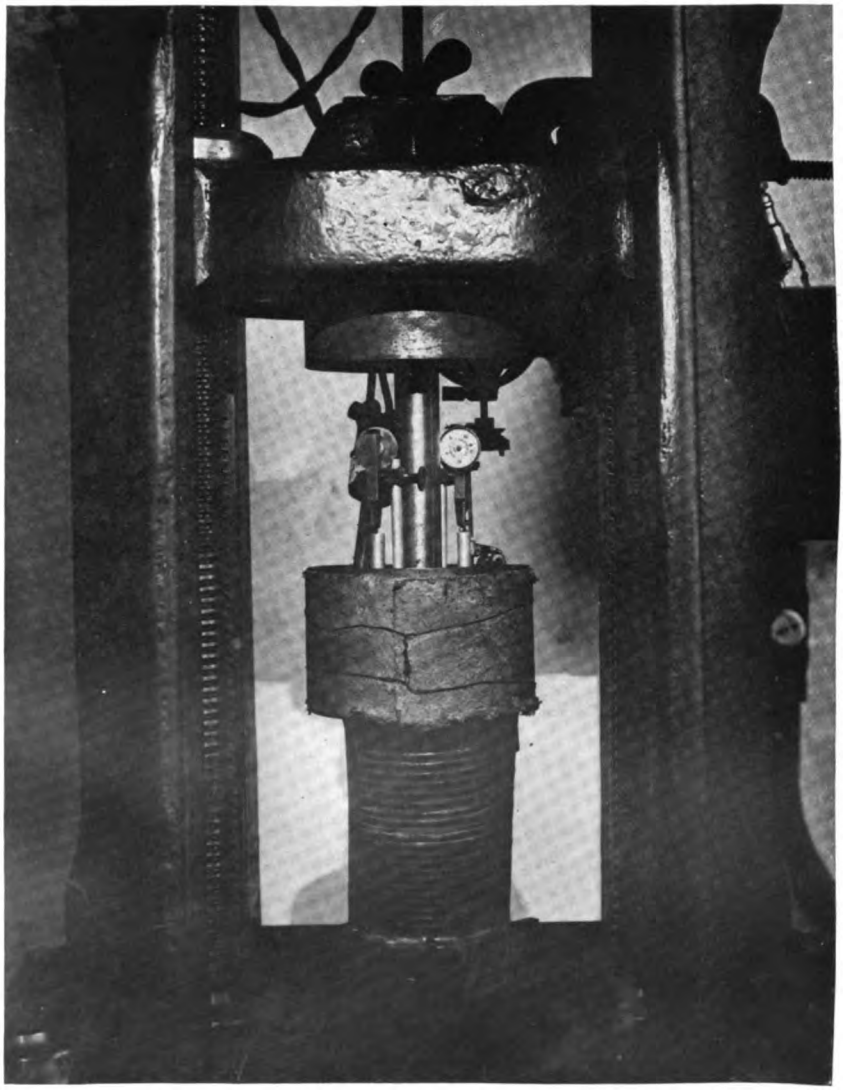

FIG. 4.—*Specimen with compressometer and heating bath assembled in testing machine*

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ing liquid (glycerin was used) with a base made to fit on the spherical seat of the Brinell machine and a post on the inside to support the specimen away from the bottom and permit good circulation of the liquid around it. The bath was stirred with a small motor-driven propeller and was heated by a small resistor placed on the bottom of the container. During test the entire specimen was submerged, the Brinell ball also being completely immersed. Sufficient time was always allowed for the specimen to reach the temperature of the bath, this having been previously determined by inserting a thermocouple in a specimen and noting the time elapsed between the placing of the specimen in the bath and when the center reached the temperature of the bath.

V. PRELIMINARY TESTS

It is well known that the pouring temperature of a bearing metal, all other conditions being constant, has a marked influence on the mechanical properties. In view of this fact, preliminary to any test at elevated temperatures, the effect of pouring temperature on the compressive strength at room temperature was determined for alloys Nos. 1, 3, 4, and 5. The results obtained from compression tests are given in Table 3.

TABLE 3.—Effect of Pouring Temperatures on Yield Point and Ultimate Strength in Compression of Various Alloys

Alloy No.	Pouring temperature	Yield point	Ultimate strength
	°C	Lbs./in.²	Lbs./in.²
1	400	3750	12 940
	446	4000	12 855
	495	3500	13 500
3	390	3500	15 830
	445	4250	16 435
	500	4000	15 830
4	300	5000	14 015
	350	4250	13 685
	400	4750	13 635
5	300	3250	13 840
	356	3750	15 020
	404	3250	15 245

In results reported in this paper the yield point was adopted arbitrarily as at 1/8 of 1 per cent reduction of the gage length. The ultimate strength was arbitrarily chosen as the unit load necessary to produce a deformation of 25 per cent of the original

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length of the test specimen. The reasons for selecting these values will be discussed later.

From a comparison of the results of Table 3 and the pouring temperatures suggested in the tentative specifications of the A. S. T. M.[5] the following temperatures were used in casting test specimens for all further tests.

	Pouring temperature, °C
No. 1	440
No. 2	440
No. 3	440
No. 4	345
No. 5	325

VI. ELEVATED-TEMPERATURE COMPRESSION TESTS

Stress deformation curves were taken on all five alloys at room temperature (20–30° C), 50, 75, and 100° C. At least two specimens were tested under each condition to provide a check.

Representative stress-strain curves at the four temperatures of each alloy except No. 2 are given in Fig. 5. These show the type of stress-deformation curve obtained with the apparatus described in this paper and also show very clearly the marked change in the compressive strength of the alloys with increasing temperatures.

On the plot a "dial unit" is equivalent to 0.00087 inch and is the algebraic mean of the total deformation shown by the individual dials for any given load.

A study of the curves shows that it is practically impossible to pick out a limit of proportionality as ordinarily determined by noting the departure of the stress-deformation curve from a straight line, and, further, we know that the finer the measurement the lower will be this point. An arbitrary yield point was therefore determined upon. After comparing the yield points indicated by several values of percentage reduction of gage length the value of ⅛ of 1 per cent of the gage length (0.00125 inch) was adopted for purposes of comparison, as it generally seems to coincide with the first marked yielding of the specimens tested. This value 0.00125 inch) is practically equivalent to 1.5 division on the dial or "dial units" used on plot (Fig. 5).

When soft metals of this type and size of test specimens are compressed they do not eventually shear but continue to flatten

[5] See footnote 2.

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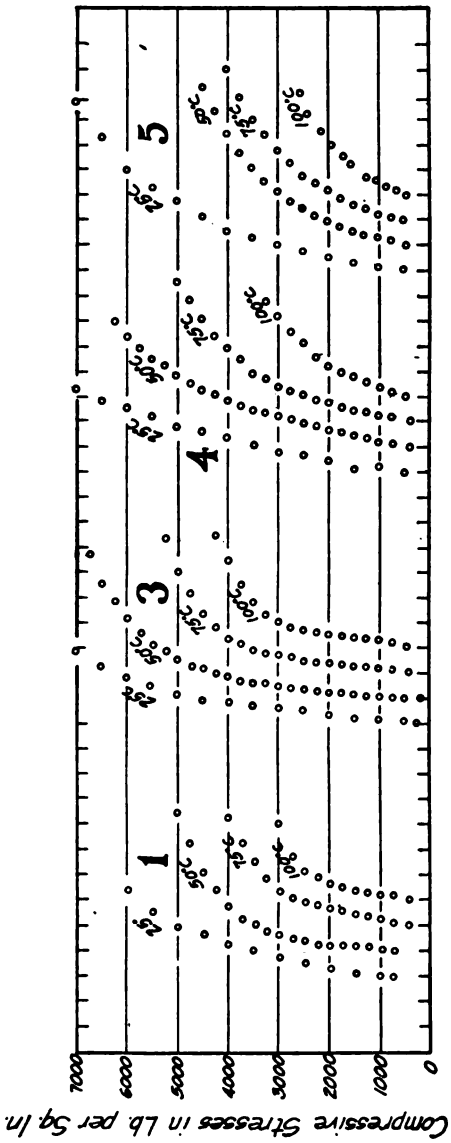

FIG. 5.—*Representative stress-strain curves of alloys Nos. 1, 3, 4, and 5 (see Table 2) at 25, 50, 75, and 100° C*

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out with increasing loads, so it is necessary to adopt some arbitrary values for ultimate strength which will at least be comparable among themselves. A reduction of 25 per cent of length was chosen in this investigation as at this value in all cases the load had become nearly constant for increasing deformation. In the case of high lead alloys the load generally reached a maximum value and then fell off before the 25 per cent reduction was reached. In these cases the maximum load was recorded. The values of yield point and ultimate strength thus obtained are given in Table 4.

TABLE 4.—Yield Point and Ultimate Strength in Compression at Elevated Temperatures

Alloy No.	Property	Values in pounds per square inch at—			
		25° C	50° C	75° C	100° C
1	Yield point	4 400	3 800	3 150	2650
	Ultimate strength	12 850	10 400	8 450	6950
2	Yield point	6 250	4 850	4 000	2850
	Ultimate strength	15 175	11 850	9 400	6825
3	Yield point	5 750	5 000	4 250	3350
	Ultimate strength	16 425	12 175	10 100	7725
4	Yield point	4 700	3 650	2 900	2150
	Ultimate strength	13 685	10 035	7 845	6045
5	Yield point	3 750	2 650	2 250	1550
	Ultimate strength	15 020	11 275	7 920	4770

VII. ELEVATED TEMPERATURE BRINELL TESTS

The Brinell hardness of alloys Nos. 1, 3, 4, and 5 was determined at room temperature, 50, 75, and 100° C. The values obtained are given in Table 5.

TABLE 5.—Brinell Hardness at Elevated Temperatures

Alloy No.	Brinell hardness numeral at—			
	25° C	50° C	75° C	100° C
1	[a] 17.2 (28.6)	13.8	11.1	8.2 (12.8)
3	22.3	18.2	14.8	11.3
4	22.4	15.8	11.3	7.5
5	19.7 (19.5)	16.8	11.4	8.2 (8.6)
2	22.3 (28.3)			

[a] A. S. T. M. specifications (see footnote 2) give the values shown in parentheses.

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VIII. DISCUSSION OF RESULTS

1. COMPRESSION TESTS

For greater convenience of comparison the yield points of the four alloys are plotted against temperature in Fig. 6*b*.

As one would expect from the composition, the yield point of alloy No. 3 is considerably higher at all temperatures than the other alloys. The yield point of No. 3, however, falls off more

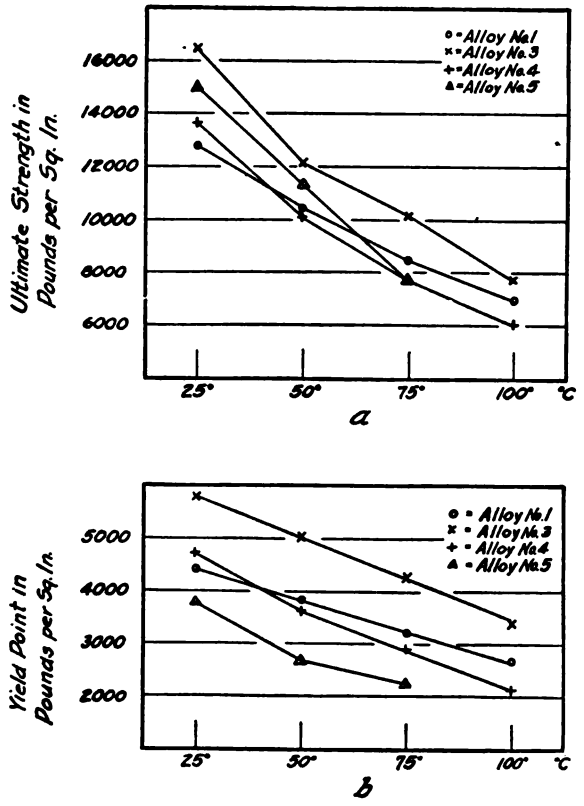

FIG. 6 *a* and *b*.—*Curves showing effect of temperature on yield points and ultimate strength of alloys Nos. 1, 3, 4, and 5*

rapidly than No. 1 with increasing temperatures. The points in both these cases appear to lie on a straight line. This is not the case with alloys Nos. 4 and 5, which contain lead. For both of these alloys the yield point seems to drop off more rapidly at first, between 25 and 50° C. It is significant to note that while the yield point of alloy No. 4 is higher than No. 1 at room temperature, it is lower at 50° C and decreases at a more rapid rate between 25 and 100° C than does No. 1, and that the yield point of tin-base alloys is higher at all temperatures above 50° C.

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The yield point of alloy No. 2, curves of which are not given, is slightly higher at room temperature than that of No. 3, but at 50° C its yield point is slightly less than No. 3, and so, if the bearing heats to 50° C or over, any advantage gained by using No. 2 alloy in a bearing is lost in so far as the yield point is concerned.

There are given in Fig. 6*a* curves showing the variation of the ultimate strength with the temperature. Here, as with the yield point, alloy No. 3 has the maximum value throughout the temperature range, and alloys Nos. 1 and 3 maintain their strength better, having a higher ultimate strength at temperatures above 60° C than either alloys Nos. 4 or 5, which contain lead, even though the ultimate strength of No. 1 at room temperature is less than that of Nos. 4 or 5. The ultimate strengths of the four alloys at 100° C stand in the same relation to each other as their respective yield points.

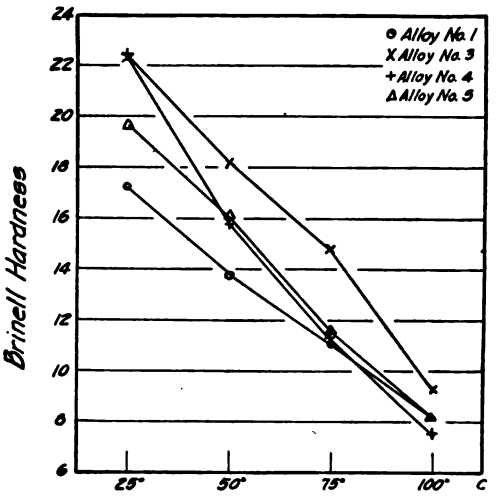

FIG. 7.—*Curves showing relation of Brinell hardness to temperature for alloys Nos. 1, 3, 4, and 5*

2. BRINELL HARDNESS TESTS

It is noted that the Brinell hardness values obtained for the tin-base alloys are considerably lower than those usually given. This difference may be due to the small percentage of impurities in the alloys used in this investigation as compared with similar alloys as ordinarily prepared.

Curves showing the variation of the Brinell hardness with temperature are given in Fig. 7. Here, again, alloy No. 3 has a maximum value throughout the temperature range. There is no evident relation between the relative magnitude of either the ultimate strength or yield point and the Brinell hardness.

The hardness of alloys Nos. 4 and 5, however, drops off very rapidly with increasing temperature, while Nos. 1 and 3 maintain their hardness, both having a greater hardness at 100° C than No. 4, and No. 1 having the same value as No. 5 at this temperature.

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IX. EFFECT OF PROLONGED HEATING AT 100° C

Oftentimes a babbitted bearing which has given good service will for no apparent reason gradually become soft and "wipe out." As a working hypothesis it was thought that this failure with age might be due to softening from prolonged heating causing an annealing action. In order to determine the validity of this tentative hypothesis, compression specimens of alloys Nos. 1, 3, 4, and 5 were heated in an oil bath for from one to six weeks at temperatures between 90 and 100° C. They were then tested at room temperature with the results given in the following table:

TABLE 6.—Effect of Prolonged Heating on the Yield Point in Compression

Days heating at 100° C	Yield points of the various alloys			
	Alloy No. 1	Alloy No. 3	Alloy No. 4	Alloy No. 5
	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²
0	4550	5750	4650	3750
7	4500	5500	.....	3450
14	4600	5800	4250	3200
28	5025	5650	4750	[a] 2800
42	4900	5950	4850	3150

[a] One specimen only. All other values are the average of two specimens.

A study of the above table indicates that for alloys Nos. 1, 3, and 4, heating at 100° C for 42 days has no appreciable effect on the value of the yield point when the specimens are cast in the manner indicated. For alloy No. 5, however, there is a very evident decrease in the value of its yield point with the prolonged heating which, however, evidently takes place in the first two weeks of the heating.

X. EFFECT OF SMALL PERCENTAGES OF LEAD

The specifications for high-grade, tin-base alloys such as Nos. 1, 2, and 3 call for a low lead content generally not to exceed 0.35 per cent.

Many believe, and one investigator [6] has presented experimental evidence, that percentages of lead even up to 5 per cent are not harmful but possibly beneficial. The authors have therefore investigated the effect of small percentages of lead on the yield point and ultimate strength of No. 2 alloy at room temperature and at 75° C.

[6] Jesse L. Jones; see footnote 1.

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The alloys were prepared by adding metallic lead to the No. 2 babbitt in amounts shown by the chemical analysis given, together with the yield points, in Table 7.

TABLE 7.—Effect of Lead on Compressive Strength

Percentage of lead	Yield point at—		Ultimate strength at—	
	25° C	75° C	25° C	75° C
	Lbs./in.²	Lbs./in.²	Lbs./in.²	Lbs./in.²
0.00	6150	4000	15 175	9 395
0.26	5850	3700	15 640	10 010
0.51	5750	3300	14 025	9 765
1.01	6300	.....	.....	.....
1.25	6000	4100	16 380	10 600
5.04	5850	3850	15 330	9 725

The addition of amounts up to 5 per cent of lead to this babbitt seems to have no very appreciable effect on its mechanical properties in compression at room temperature or at 75° C under the conditions of test used. The authors think, however, that these tests should not lead to an increase in the lead content tolerance in tin-base bearing metal specifications until much more work is done along this line and particularly to determine the possible effect of small percentages of lead on the resistance to repeated impact.

XI. SUMMARY AND CONCLUSIONS

An apparatus is described for determining the yield point and ultimate strength of white metal bearing alloys at temperatures up to 100° C. A new design of heating apparatus is described for determining the Brinell hardness of metals at temperatures up to 100° C.

The results of compression tests and Brinell hardness tests at temperatures up to 100° C are given for five typical white metal bearing alloys, including three tin-base alloys, one lead-base alloy, and one intermediate alloy, which show that the tin-base alloys maintain their properties better at elevated temperatures than the lead-containing alloys.

Results of tests are given which indicate that the addition of amounts up to 5 per cent of lead in a high-grade babbitt does not affect the yield point or ultimate strength at 25 or 75° C.

The yield point of tin-base alloys is not affected by heating for six weeks at about 100° C, but the yield point is lowered in the lead-base alloy by heating for two weeks at about 100° C.

WASHINGTON, November 6, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 189

METHOD FOR DIFFERENTIATING AND ESTIMATING UNBLEACHED SULPHITE AND SULPHATE PULPS IN PAPER

BY

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APRIL 4, 1921

PRICE, 5 CENTS
Sold only by the Superintendent of Documents, Government Printing Office, Washington, D. C.

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A METHOD FOR DIFFERENTIATING AND ESTIMATING UNBLEACHED SULPHITE AND SULPHATE PULPS IN PAPER

By R. E. Lofton and M. F. Merritt

ABSTRACT

The purpose of this paper is to fill a need felt especially by paper chemists and analysts for a rapid and reliable method of distinguishing between and of making an approximately correct quantitative determination of mixtures of unbleached sulphite and sulphate pulps. This paper gives briefly the basic differences in the manufacture of the two pulps, and contains a concise review of the methods that have been recommended from time to time for distinguishing between unbleached sulphite and sulphate pulps. It gives the procedure followed in developing a new and comparatively rapid method for distinguishing between these pulps, and also gives some of the more important experiments carried out with various stains during this investigation. The method of preparing the new stain and the method of procedure for differentiating between unbleached sulphite and sulphate fibers is described in detail, and tables showing the results of quantitative microscopical analysis of mixtures of these fibers stained by the new method are given.

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I. INTRODUCTION

The purpose of this publication is to give a review of the various methods proposed for distinguishing between unbleached sulphite and sulphate fibers, and especially to describe the development and application of a new method for accomplishing this result.

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That there is and has existed for some time a demand for a quick and certain method for differentiating between these two pulps is indicated by the various attempts that have been made from time to time to develop such a method. To be satisfactory, any method proposed must be practical as well as certain—that is, it must not require any expensive apparatus, or special experience, or tedious and lengthy manipulation—but it must be such as may be carried out in a few minutes time by any one ordinarily familiar with the microscopical examination of paper-making fibers.

Such a method would be of service to pulp manufacturers and jobbers, to manufacturers of sulphite and sulphate papers, and to the retailers and consumers of sulphite and sulphate papers in aiding them to determine whether they are getting what they desire. It is useless for a jobber or a consumer to specifically order an all or part sulphate wrapping paper unless there is some means of determining whether the article ordered is being furnished.

The authors acknowledge the assistance of M. B. Shaw, who made a large number of fiber estimations, the results of which appear in Tables 3, 4, and 5, and who gave assistance as to methods of preparing the stains.

II. THEORETICAL CONSIDERATIONS INVOLVED

1. FUNDAMENTAL DIFFERENCES BETWEEN THE TWO PULPS

The difficulties to be overcome in developing a method for distinguishing between unbleached sulphite and sulphate pulps are due to the similarity of the pulps. Both are made, for the most part, from the same raw material, with but two or three exceptions. Spruce, hemlock, balsam fir, yellow pine, tamarack, and white fir are used in making both pulps. In addition to these woods, jack pine and cypress are often used in the sulphate process. Yellow pine is more generally used in the sulphate than in the sulphite process, because the former process is better adapted to woods rich in rosin and oil. The woods generally used in foreign countries are of the same kinds as those used in the United States and Canada except that black spruce is frequently used in Finland, Norway, and Sweden.

Since the raw material from which these pulps are made is in general the same, the only distinguishable differences between the pulps are to be found in the two different digesting processes employed in their manufacture.

Sulphite pulp is made by cooking wood chips in a solution of bisulphite of calcium, or of calcium and magnesium. This solution

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has an acid reaction. The chips are cooked under a steam pressure of from 60 to 80 pounds for from 8 to 20 hours. The process was invented by B. C. Tilghman, of Philadelphia, who took out the first United States patent in 1867.

Sulphate pulp, frequently called "Kraft" (a German word meaning "strength"), is made by cooking wood chips in a solution the chief ingredient of which is sodium sulphide. The sodium "sulphate" added from time to time is reduced to "sulphide" during the preparation of the cooking liquor. The reaction of this solution is alkaline. In this process the chips are cooked under a steam pressure of about 100 pounds for from 2 to 7 hours. This process was invented by Carl F. Dahl, of Danzig, Germany, about 1883, and was introduced into America in 1907, when the Brompton Pulp and Paper Co. set up a sulphate mill in Canada.

Unbleached sulphite pulp is used in the manufacture of wrapping paper and bag stock, of many so-called Kraft and manila papers, of twines used in tying bundles and in making paper rugs, onion and potato sacks, and in any pulp or paper product where strength is the chief consideration. Sulphate pulp has a considerably darker color than sulphite, for which reason the latter is often colored in the process of being made into paper to resemble sulphate pulp. In general, sulphate pulp may be used wherever it is permissible to use unbleached sulphite. In cases where the greatest possible strength is required, sulphate is used instead of sulphite, since it is somewhat stronger. The cost of sulphate, due to a more expensive process of manufacture, is somewhat more than that of sulphite pulp.

2. POSSIBLE BASES OF DIFFERENTIATION

Commenting on the problem, C. W. Schwalbe stated[1] in 1914 that there was no simple method known at that time for distinguishing between unbleached sulphite and sulphate pulps; also, that it is difficult to develop such a method because the pulps are so closely related. Schwalbe recognized, too, as do others who have made a study of the two pulps, that there are two differences between them which may be used as a basis on which to develop reactions which will differentiate them, namely, (1) the difference in the amounts of incrusting or ligneous material, and (2) the different chemical changes which have been brought about by the different digesting processes.

[1] Testing Methods for Sulphite and Sulphate Cellulose in Paper, Pulp and Paper Magazine, p. 21; Jan. 2, 1914.

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3. DIFFERENT AFFINITIES OF PULPS FOR DYES

It is pretty generally known by persons who have made any analytic study at all of paper-making pulps that those pulps differing in the degree of cooking, or of bleaching, or both, usually have quite different affinities for various dyes and stains. The dye most generally used to show the properties of different pulps in this respect is malachite green, a basic aniline or coal-tar dye which has a great affinity for highly lignified fibers and very little or no affinity for pulps and fibers freed from incrusting matter. The result of this property is that, when malachite green alone is used to stain a pulp composed of a mixture of fibers having a high content of lignin and of other fibers more thoroughly digested, one gets a range of shades varying from deep green to very light green, or, perhaps, to the entire absence of color in the case of fibers that are completely freed from incrusting matter. Although there may be no sharp color line of demarcation to set off any one group of fibers against any other, results of this kind may serve the purpose of determining the thoroughness or the uniformity of the cooking or of the bleaching action. But if this or a similar dye or stain is followed by one of a contrasting color, as recommended by Siebert and Minor,[3] it will sometimes be found possible to separate quite distinctly a mixture of two different pulps, as, for example, a well-cooked from a slightly-cooked pulp, or a well-bleached from a poorly-bleached pulp, or a chemical from a mechanical pulp. These authors, however, recommend that malachite green be followed by an acid dye, while in the method described below both of the dyes used are basic.

Whether the different affinities of the two pulps for various dyes and stains are due to different chemical properties of the pulps brought about by the different cooking methods, or whether they are due merely to the different amounts of incrusting matter remaining in the sulphite and sulphate pulps, is a question on which cellulose chemists do not agree.

III. METHODS PROPOSED BY EARLIER EXPERIMENTERS

In studying this problem the available methods proposed from time to time by others who have experimented along this line were tried. Below is given a description of each method, together with a brief statement of results obtained by the writers in trying the method.

[3] The Differentiation of Sulphite Pulps, Paper, 25, No. 21; Jan. 28, 1920.

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1. KLEMM'S METHODS

Klemm is given credit for developing two methods, in one of which use is made of malachite green alone, and in the other use is made of both malachite green and rosaniline sulphate.

Klemm's malachite green method is described by R. W. Fannon [3] as follows:

> The reagent consists of a saturated solution of malachite green, a coal-tar dye, to which 2 per cent acetic acid has been added. A portion of pulp is treated with a few drops of the reagent sufficient to soak it, and the excess blotted up. The fiber is then examined microscopically. Unbleached sulphite is colored full green; sulphate is colored light green.

The results obtained by using this method were not satisfactory, since there is no sharp or distinct line of demarcation between the two pulps, some of the more deeply colored sulphate fibers showing a deeper green than some of the lighter colored sulphite fibers. These results are in harmony with the statements made above regarding the use of malachite green in staining various pulps and fibers.

Schwalbe mentions another procedure,[4] a malachite green and rosaniline-sulphate method, which he also calls Klemm's method. No details as to making up the dye solutions or method of applying them are given.

It was found, however, that a fairly good differentiation between the two pulps is had if they are stained for about two minutes with a one-half per cent aqueous solution of malachite green, rinsed with water, then stained with a solution of rosaniline sulphate acidified with sulphuric acid. The same results may also be obtained if these two stains are compounded in the right proportions before being applied to the pulps.

When the chemical composition of rosaniline sulphate is compared with that of magenta, or fuchsine, one of the dyes recommended below, it is evident that they are very closely related, and that the color radical is the same in both. An insight into the constitution of these compounds may be had from any good treatise on organic chemistry.

Pararosaniline and rosaniline [5] are the bases of the fuchsine dyes. Both are triacid bases, stronger than ammonia. The basic fuchsine or magenta dyes are formed by treating these bases with various acids, particularly hydrochloric and acetic,

[3] An effort to find a simple means of differentiation between sulphite and sulphate pulp. (A thesis prepared at the University of Maine, June, 1916.)

[4] Pulp and Paper Magazine, p. 21; Jan. 1, 1914.

[5] A. Bernthsen, Organic Chemistry, translation by J. J. Sudborough; New York, 1912.

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water being eliminated during the reaction. The formula of rosaniline is

$$C(OH)\left\langle\begin{matrix}C_6H_4.NH_2\\C_6H_4.NH_2\\C_6H_3(CH_3).NH_2\end{matrix}\right\rangle \text{ or } C_{20}H_{21}N_3O.$$

By treating with hydrochloric acid, the following reaction takes place:

$$\underset{\text{Rosaniline}}{C_{20}H_{21}N_3O} + \underset{\substack{\text{Hydro-}\\\text{chloric}\\\text{acid}}}{HCl} = \underset{\substack{\text{Rosaniline}\\\text{hydrochloride}}}{C_{20}H_{20}N_3Cl} + \underset{\text{Water}}{H_2O}.$$

From analogy, it appears that a similar reaction takes place when rosaniline is treated with sulphuric acid, and that rosaniline sulphate is essentially a basic fuchsine or magenta dye, but with an acid radical not usually found in dyes put out under this name.

$$\underset{\text{Rosaniline}}{(C_{20}H_{21}N_3O)_2} + \underset{\substack{\text{Sulphuric}\\\text{acid}}}{H_2SO_4} = \underset{\substack{\text{Rosaniline sul-}\\\text{phate}}}{(C_{20}H_{20}N_3)_2SO_4} + \underset{\text{Water}}{2H_2O}.$$

As the coloring power, however, of these dyes lies wholly in the basic constituent, there can be no doubt but that rosaniline sulphate is essentially basic fuchsine or magenta.

One authority [6] states that basic magenta is a mixture of the hydrochloride and acetate of rosaniline and pararosaniline; according to another, it is a mixture of the hydrochlorides of rosaniline and pararosaniline. The formula of pararosaniline is

$$C(OH)\left\langle\begin{matrix}C_6H_4.NH_2\\C_6H_4.NH_2\\C_6H_4.NH_2\end{matrix}\right\rangle \text{ or } C_{19}H_{19}N_3O.$$

The reaction of this base with acids is like that of rosaniline.

The malachite-green and rosaniline-sulphate stain, however, does not give as brilliant color contrast between the two kinds of fibers. This advantage of the use of basic fuchsine over that of rosaniline sulphate is probably explained by the greater solubility of basic magenta as compared with that of rosaniline sulphate, since it was found by test that basic magenta is much more soluble in water than is rosaniline sulphate. This fact is in harmony with the known properties of the salts of these acids.

[6] Thomas H. Norton artifical dyestuffs used in the United States, Special Agent Series, No. 121, Bureau Foreign and Domestic Commerce, Washington D. C. Ira Remsen, Organic Chemistry, Fifth revision; New York, 1909.

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2. SCHWALBE'S METHODS

Schwalbe also describes[7] two methods of analysis, in one of which aqueous solutions of ferric chloride and potassium ferrocyanide are used, and in the other solutions of a copper salt and an organic dye. The former method is also briefly outlined by Fannon.[8]

The ferric-chloride and potassium-ferrocyanide method consists in extracting the pulps for about 30 minutes in alcohol and ether to remove rosin, treating with a 0.05 *N* solution of ferric chloride at from 60 to 80° C for about 30 minutes, or until all the fibers settle to the bottom of the container, washing with distilled water, and treating with a 1 per cent solution of sulphuric acid, to which is then added from four to eight drops of a 2 per cent solution of potassium ferrocyanide. The container and contents are then placed on a water bath and kept at a temperature of from 60 to 80° C for from 5 to 10 minutes. The pulps are then washed and examined under the microscope. Sulphite fibers are colored a deep blue, and sulphate fibers a much lighter blue.

This method did not give results that would enable a distinction to be made in all cases between the two pulps, nor to permit estimations in any case. Moreover, the process is too long and tedious to be practical. However, since Schwalbe so highly recommends this method, it is thought probable that better results could be obtained if enough time should be spent in studying and experimenting with it.

By the copper method the pulps are boiled in a solution of a copper salt such as copper sulphate, washed, then treated with a solution of an organic dye such as benzopurpurine 10B. The benzopurpurine treatment develops in some classes of fibers an intensely blue color, according to Schwalbe, due to the formation of a so-called copper lake with the copper salt retained by the fibers.

This method was not experimented with, since Schwalbe himself condemns it as being too involved and uncertain in results, especially with pulps so similar in their properties as unbleached sulphite and sulphate.

---

[7] Pulp and Paper Magazine, p. 21; Jan. 1, 1914.
[8] Thesis referred to above.

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3. FANNON'S METHODS

Fannon [9] found that a saturated aqueous solution of rosaniline sulphate, to which is added 1 to 3 per cent of alcohol and enough sulhpuric acid to cause the solution to take on a violet shade, is a satisfactory stain. He used two methods of applying this stain and examining its staining action:

In the first method, two or three drops of the solution were allowed to fall in the same spot onto the samples of sulphite and sulphate pulps tested, and the color reactions observed with the unaided eye. The unbleached sulphite pulps showed "a deep bluish center with a yellow ring surrounding," while the sulphate sample gave "a deep red coloration." "From 7 to 15 minutes after the stain was dropped on, the distinctive colorings were evident. After an hour or so the colors faded."

In the second method, "mixtures of sulphate fibers and sulphite fibers were made, stained with the last-named solution. The different fibers developed their respective colorations, and under the microscope estimations of each were possible."

This stain was not found satisfactory when used according to either method recommended, because the color differences shown by the two pulps are not sharp enough to enable one to detect without doubt even the presence of small percentages of one pulp when mixed with the other.

IV. THE MALACHITE-GREEN AND FUCHSINE METHOD

1. SOURCES OF MATERIALS USED

The samples of pulps and papers used in this investigation consisted of 188 pieces collected from a number of different firms, and represent pulps manufactured in all parts of this country, and in at least four foreign countries, Canada, Finland, Norway, and Sweden. All possible information was obtained regarding these samples, such as the kind of wood used and in what locality grown, the degree of cooking, etc.

The dyes and stains used during the earlier part of this investigation were obtained chiefly from the chemical division of this Bureau. Most of them had been in stock for some time, and all, or nearly all, were of German manufacture. But after the method given below had been pretty thoroughly worked out, samples of the two dyes used in this method were obtained from different sources in this country in order to determine whether the different makes of dyes were equally suitable for the purpose.

[9] Thesis referred to above.

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2. METHOD OF ATTACK AND EARLIER EXPERIMENTS

The method of conducting the investigation was to try out on the two kinds of pulps the reaction of various biological stains and dyes recommended by authorities on microscopical methods, and also other stains and dyes that were suggested in one way or another. Most of the experimenting was carried on with red, green, and blue dyes, since dyes of these colors are usually the most brilliant and positive in their coloring action. The few yellow dyes that were tried did not give sufficiently brilliant color effects on the fibers, and little work was done with them.

In making this investigation by what may be properly called the empirical method, a great number of tests were necessarily made that are relatively valueless and therefore are omitted from this publication. Those which may be of value to others who are experimenting along the same line are given in Table 1.

TABLE 1.—Interesting Experiments Made and Stains Used with the Color Reactions on Sulphate and Sulphite Fibers

[Ext. alc.=extracted with alcohol; aq.=aqueous; so'.=solution.]

Serial No.	How stained	Action on—		Remarks
		Sulphate	Sulphite	
2	Safranin in equal parts water and alcohol, rinsed with water, Delafield's hematoxylin.	Colorless to faint reddish brown.	Pink to light magenta.	Color difference quite noticeable.
6	Ext. alc., boiled 1 minute, ¼ per cent aq. sol. malachite green, rinsed with water, rosaniline sulphate and little sulphuric acid.	Clear blue, bundles green.	Pale gray to lavender.	Differentiation good.
7	Ext. alc., boiled 1 minute, ¼ per cent aq. sol. malachite green, rinsed with water, eosin in equal parts alcohol and water.	Dark blue.....	Colorless to faint purple.	Pits of sulphite fibers colored green.
13	Ext. alc., heated to 100°C in ¼ per cent aq. sol. malachite green, rinsed with water, 1 per cent aq. alc. sol. safranin.	Light to dark red.	Light to dark red.	Do.
17	Boiled 1 minute in 1 part rosaniline sulphate sol., then 1 part 1 per cent aq. sol. malachite green added, lightly rinsed with water.	Blue.....	Purple.....	Good differentiation.
44	Ext. alc., boiled in water, acidified solution 50 per cent alcohol, Delafield's hematoxylin 10 to 15 minutes, rinsed with water, ½ per cent sol. congo red 1 minute, rinsed with water.	Red.....	.....do.....	Fair differentiation.
45	Ext. alc., boiled in water, acidified solution 50 per cent alcohol, Delafield's hematoxylin left on until air dry, rinsed with water, ½ per cent sol. congo red 1 minute, rinsed with water.	.....do.....	Purple with little red.	Do.

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TABLE 1.—Interesting Experiments Made and Stains Used with the Color Reactions on Sulphate and Sulphite Fibers—Continued

Serial No.	How stained	Action on—		Remarks
		Sulphate	Sulphite	
57	Ext. alc., washed with 1 per cent aq. sol. tannic acid, then equal parts 1 per cent aq. solutions acid fuchsine and malachite green 3 minutes, rinsed with water.	Blue.....	Purple.....	Good differentiation.
79	Ext. alc., boiled in water, dried with filter paper on slide, then 50 parts each of 1 per cent aq. solutions magenta and malachite green and 1 part 1 per cent aq. sol. tannic acid 2 minutes, quickly rinsed in 50-50 aq. alc. sol. slightly acidified with HCl, rinsed with water.	Greenish blue.....	....do.....	Differentiation excellent.
82	Boiled in water, ½ per cent aq. sol. malachite green and aq. sol. rosaniline sulphate and little H_2SO_4, all left on fibers 2 minutes, then rinsed with water.	Blue.....	.....do.....	Color difference not as decisive as when magenta is used in place of rosaniline sulphate.
84	Slide as made up by No. 79 examined under microscope in polarized light, with both parallel and crossed nicols.	Various colors...	Various colors.	No distinguishing properties appeared.
85	Boiled in water, pulped and examined under microscope in polarized light with both parallel and crossed nicols.	Yellow, orange, and blue.	Yellow, orange, and blue.	Do.

Some of the pulps were extracted with alcohol to remove resins, and this fact is indicated in each case in the table. The pulps were then rinsed and cooked, at first, in clear water, and pulped, no chemical being used in the cooking process, as it was thought that any chemical treatment might possibly interfere with the staining action to follow. Experiments that were conducted later, however, indicated that cooking the sample of paper in a one-half per cent aqueous solution of caustic soda does not have any effect on the action of the dyes. The alcohol used at various times and for various purposes, as indicated, was in all cases ethyl alcohol. The color appearances and other characteristics noted are those which appeared under the microscope.

3. DETAILED METHOD OF USING

The stain which was found to be most satisfactory in differentiating between unbleached sulphite and sulphate pulps or fibers was a mixture of one part of a 2 per cent aqueous solution of malachite green and two parts of a 1 per cent aqueous solution of

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basic fuchsine, or magenta. The solutions were made up according to the following formulas, kept in tightly stoppered separate bottles, and mixed only when wanted for use:

A—Malachite green	2 g
Distilled water	100 cm^3
B—Basic fuchsine	1 g
Distilled water	100 cm^3

Since there is considerable variation in the quality of dyes from various sources, it is not to be expected that any given combination of dyes or method of procedure will best fit all cases; it is, indeed, more than probable that the compound stain will have to be modified somewhat as to its two components, depending on the source of the dyes.

After this stain, therefore, has been made up according to formula, it will be necessary to test it out on samples of sulphite and sulphate fibers. To do this, samples of unbleached sulphite and sulphate pulps should be prepared and a few fibers of each placed on a slide, care being taken not to get the two samples mixed. The fibers are then dried and stained, as directed below, and then examined under the microscope. All the sulphate fibers should have a blue or blue-green color, and all the sulphite fibers should have a purple or lavender color. If any purple fibers appear in the sulphate pulp this indicates that too much fuchsine is present in the combination, and a little more malachite green solution must be added to counteract this effect. If, on the other hand, some of the sulphite fibers show green or blue, there is too much malachite green in the combination, and more fuchsine solution must be added. Of course the analyst must be sure that he is using authentic samples of the two pulps for this test. When tested out in this manner and the proper combination found, the stain is ready to be used on unknown combinations of fibers containing either unbleached sulphite or sulphate, or both.

A mixture of one-half sulphite and one-half sulphate may also be used to test out the stain, the proper combination for the stain being indicated when one-half of the fibers are colored blue, and the other half purple.

The stain should not be used for more than a few hours after being compounded and should be made up anew at least each day.

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The method of preparing the samples of pulps or papers for staining, and of applying the stain, is as follows:

The sample is boiled for a few minutes in water or in a one-half per cent aqueous solution of sodium hydroxide, and the fibers are thoroughly disintegrated by shaking in a test tube or other receptacle about half filled with water, glass beads being added if the fibers can not otherwise be separated. Several fibers are then removed by means of a teasing needle, or preferably by means of a glass tube [10] about seven thirty-seconds of an inch in diameter, placed on a microscope slide, and dried by the use of hard filter or blotting paper. Two or three drops of the compound stain are then placed on the fibers by means of a suitable dropper or a pipette and allowed to remain 2 minutes, during which time the fibers are being teased apart and moved about in the stain on the slide. This teasing is necessary in order that the stain may have equal opportunity to act on all the fibers. At the end of 2 minutes the excess stain is removed with three or four thicknesses of hard filter paper, and the fibers treated with three or four drops of a weak aqueous solution of hydrochloric acid, made by adding 1 cm^3 of concentrated acid (sp. gr. 1.19; HCl 37 per cent) to 1 liter of distilled water. The acid solution is allowed to remain on the slide for from 10 to 30 seconds, during which time the fibers are teased and moved about rapidly. Following this, the excess acid solution is removed with filter paper, three or four drops of distilled water applied, the fibers quickly teased about, and the water absorbed with filter paper. If all the excess stain has been removed from the slide at this point, a drop or two of water may be added, the fibers spread about on the slide, and a cover glass placed over them. But if too much stain remains on the slide at this point, it will be necessary to rinse again with distilled water before applying the cover glass. After the cover glass has been placed in position the fibers are ready for examination under the microscope.

4. ESTIMATING PERCENTAGES

The color contrast not only enables one to detect the presence of one or both of these fibers, but is sharp enough to enable one, after some practice, to make an approximately correct estimate of the percentages of each of these fibers present.

[10] F. C. Clark, Paper Testing Methods, Tappi Publishing Corp., New York N. Y.; 1930.

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In order to get a practical idea as to what could be accomplished in estimating the percentages of sulphite and sulphate fibers when stained as directed, three persons, all of whom had had experience in estimating the percentages of fibers stained with the zinc-chloride and iodine stain, made a number of estimations on known mixtures of these two pulps.

Seven mixtures, made from representative samples of sulphite and sulphate pulps, were prepared by weighing on a chemical balance the proper proportions of each pulp, the weights of the two components totaling 30 g in each case. The pulps were then thoroughly mixed by agitating in a tight container with considerable water. The series made up contained 20, 25, 40, 50, 60, 75, and 80 per cent of unbleached sulphite. A representative portion of each member of this series was placed in a container and labeled. Microscope slides were then made up from each of the seven samples, each slide being given an unknown mark of identification, and handed over to the analysts for their estimations. Four different series of estimates were made, in each of which a different make of American dyes was used.

V. SUMMARY OF RESULTS

In the tables below are given the results of estimates on the seven fiber mixtures, each table showing the results obtained by using dyes from one of the four sources.

TABLE 2.—Results of Analyses, Using Dyes from First Source

Number of estimations	Observer	Percentage sulphite in mixture				Error of average	Probable error (r)	Huge error (u)
		Estimated			Actual			
		Maximum	Minimum	Average				
						Per cent	Per cent	Per cent
16	A	40	20	22.8	20	+2.8	5.1	12.3
14	C	50	10	27.9	20	+7.9	8.8	21.6
16	A	50	20	33.1	25	+8.1	8.2	20.0
14	C	50	20	33.9	25	+8.9	8.7	21.2
16	A	60	25	41.6	40	+1.6	7.1	17.3
14	C	90	25	50.7	40	+10.7	12.7	30.9
16	A	75	40	51.6	50	+1.6	5.4	13.2
14	C	80	40	57.1	50	+7.1	9.5	23.1
16	A	80	60	67.2	60	+7.2	7.1	17.3
14	C	90	60	69.6	60	+9.6	8.7	21.2
16	A	90	25	73.7	75	−1.3	9.6	23.4
14	C	80	40	72.9	75	−2.1	7.8	18.9
16	A	95	75	80.6	80	+0.6	3.4	8.2
14	C	90	25	76.4	80	−3.6	10.1	24.7

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TABLE 3.—Results of Analyses, Using Dyes from Second Source

Number of estimations	Observer	Percentage sulphite in mixture				Error of average	Probable error (r)	Huge error (u)
		Estimated			Actual			
		Maximum	Minimum	Average				
						Per cent	Per cent	Per cent
16	A	40	20	24.7	20	+4.7	6.0	14.7
2	B	25	25	25.0	20	+5.0	3.4	8.2
5	C	40	20	24.0	20	+4.0	6.0	14.7
16	A	50	20	37.2	25	+12.2	10.0	24.4
3	B	40	25	35.0	25	+10.0	8.3	20.2
5	C	25	20	24.0	25	−1.0	1.5	3.7
16	A	75	25	50.0	40	+10.0	10.5	25.5
3	B	60	50	53.3	40	+13.3	9.5	23.3
6	C	75	30	47.5	40	+7.5	10.8	26.3
16	A	75	50	54.1	50	+4.1	5.4	13.2
3	B	60	60	60.0	50	+10.0	6.7	16.5
6	C	60	40	51.7	50	+1.7	6.2	15.0
16	A	80	50	65.0	60	+5.0	7.6	18.6
3	B	75	50	66.7	60	+6.7	9.1	22.3
5	C	60	50	56.0	60	−4.0	4.3	10.4
16	A	80	60	75.6	75	+0.6	4.3	10.5
3	B	80	80	80.0	75	+5.0	3.4	8.2
5	C	80	70	77.0	75	+2.0	3.0	7.4
16	A	80	75	78.4	80	−1.6	1.9	4.6
3	B	80	75	78.3	80	−1.7	2.0	4.8
4	C	80	70	73.7	80	−6.3	5.1	12.4

TABLE 4.—Results of Analyses, Using Dyes from Third Source

Number of estimations	Observer	Percentage sulphite in mixture				Error of average	Probable error (r)	Huge error (u)
		Estimated			Actual			
		Maximum	Minimum	Average				
						Per cent	Per cent	Per cent
16	A	50	20	27.5	20	+ 7.5	8.8	21.4
16	B	40	20	26.9	20	+ 6.9	7.1	17.2
16	A	75	20	32.5	25	+ 7.5	10.6	25.8
16	B	50	20	37.2	25	+12.2	11.0	27.0
16	A	60	25	43.1	40	+ 3.1	8.0	19.5
16	B	75	25	48.4	40	+ 8.4	9.9	24.3
16	A	60	20	45.3	50	− 4.7	8.1	19.9
16	B	75	40	55.3	50	+ 5.3	7.6	18.5
16	A	75	25	55.0	60	− 5.0	8.0	19.5
16	B	80	40	63.8	60	+ 3.8	8.1	19.8
16	A	80	60	74.4	75	− 0.6	4.9	12.0
16	B	80	50	70.9	75	− 4.1	6.3	15.3
16	A	80	50	75.3	80	− 4.7	6.4	15.5
16	B	80	70	78.4	80	− 1.6	2.2	5.4

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TABLE 5.—Results of Analyses, Using Dyes from Fourth Source

Number of estimations	Observer	Percentage sulphite in mixture				Error of average	Probable error (r)	Huge error (u)
		Estimated			Actual			
		Maximum	Minimum	Average				
						Per cent	Per cent	Per cent
17	A	25	20	20.6	20	+0.6	1.2	2.8
17	B	25	20	20.6	20	+0.6	1.2	2.8
17	A	40	20	28.2	25	+3.2	5.0	12.1
17	B	50	20	27.1	25	+2.1	4.8	11.8
17	A	60	25	40.6	40	+0.6	5.3	12.9
17	B	50	25	30.7	40	+0.3	3.0	7.2
17	A	60	40	48.8	50	−1.2	3.3	8.0
17	B	75	40	50.9	50	+0.9	6.8	16.4
17	A	75	25	58.8	60	−1.2	7.7	18.7
17	B	75	50	59.4	60	−0.6	4.8	11.6
17	A	80	60	74.1	75	−0.9	4.7	11.5
17	B	80	60	74.1	75	−0.9	4.7	11.5
17	A	80	75	78.2	80	−1.8	2.0	4.9
17	B	80	75	78.5	80	−1.5	1.8	4.5

The analysts, or persons who made the estimations, are referred to as "A," "B," and "C."

The error of averages is the difference between the average of all estimations for that particular case and the actual percentage of sulphite in the mixture. The average result is computed from a number of estimations in most cases; the exact number of estimations in each case is given in the tables.

The most probable error is an error of such magnitude that a single estimation has an equal chance of being either within the error or outside of it. The given errors are computed by the method of least squares according to the formula

$$r = 0.6745\sqrt{\frac{\Sigma X^2}{n}},$$

in which r is the most probable error, X is the error of one estimation, ΣX^2 is the sum of the squares of all errors of n estimations, and n is the total number of estimations made on that particular mixture by the given analyst.

The huge error is an error of such magnitude that the chances are equal that 9 estimations out of 10 will be within the error, and only 1 in 10 greater than it. These results are computed by the method of least squares, according to the formula

$$u = 1.65\sqrt{\frac{\Sigma X^2}{n}}, \text{ or } u = 2.44r,$$

in which u is the huge error.

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In no case did an individual analyst make more than one estimation on the same miscroscope slide or the same field of fibers.

It is noteworthy that the errors in the last series, or table, are much less than those in the first three series. This is no doubt due in large part to the differences between the dyes used in the first three series and those used in Table 5, since those used in the last series gave much more satisfactory color effects than those used in the first three series. This result is also certainly due in part to the greater familiarity and experience the analysts had with the stain at the time the last series was run.

There are at least two criticisms that may be offered against these estimations. The first is that the results given may not quite represent estimates on wholly unknown mixtures, since each analyst knew in what proportion each of the seven combinations in the series was made up; although, as stated above, each sample was marked in code so that the analyst could not know what combination was being examined. When a study of the individual estimates is made, it appears that this circumstance did have an appreciable influence on the estimates on the mixtures near either end of the series. But since 0 per cent and 100 per cent are limits in all mixtures, it seems probable that estimates on mixtures near these limits will always be more nearly correct than estimates on mixtures containing a higher percentage of the smaller constituent.

The second criticism is that it is not practical in most cases to make 16 or 18 different estimates on the same sample of pulp or paper. This criticism is valid, especially as applied to mill conditions; for the paper-mill chemist is usually too busy to give so much time to the study of one sample of pulp or paper. It is necessary, however, that anyone, whether novice or expert, doing work of this kind, shall become familiar by actual experience with the use and properties of any stain before reliable results in estimating percentages can be had. To get best results in estimating, it is also necessary that one keep constantly in practice, and refer often to standard or known mixtures, accurately made up, and kept always at hand. When the analyst becomes familiar with the characteristics and use of the stain described above, it is probable that estimates can be made by its use as quickly and with as much accuracy as with the zinc-chloride and iodine stain.

WASHINGTON, November 27, 1920.

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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 190

"BLACK NICKEL" PLATING SOLUTIONS

BY

GEORGE B. HOGABOOM, Electroplating Inspector
T. F. SLATTERY, Electroplater
L. B. HAM

Bureau of Standards

APRIL 4, 1921

PRICE, 5 CENTS

Sold only by the Superintendent of Documents, Government Printing Office
Washington, D. C.

WASHINGTON
GOVERNMENT PRINTING OFFICE
1921

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"BLACK NICKEL" PLATING SOLUTIONS

By George B. Hogaboom, T. F. Slattery, and L. B. Ham

ABSTRACT

To produce the so-called "Government bronze" finish on military hardware, "black nickel" plating was frequently applied. Investigation showed that for this purpose very complicated solutions were frequently employed, and at times great difficulty was encountered in producing uniform results. This paper describes the results of a few experiments on such solutions and contains recommendations regarding the composition and conditions of operation which will yield satisfactory deposits.

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1. INTRODUCTION[1]

During the war a very considerable demand arose for a black finish, the so-called "Government bronze," on military hardware and equipment. Owing to the difficulty of specifying definitely the color or other desired properties of the deposit, or the composition of solutions or methods of deposition, there was frequent uncertainty and even confusion in the various specifications and methods of inspection. In consequence numerous inquiries for information and assistance were received by the Bureau of Standards, directly and through military channels. The present paper is simply a summary of experiments and observations made and conclusions reached in the effort to answer such inquiries.

2. GENERAL PRINCIPLES

The principal formulas proposed for "black nickel" solutions[2] fall roughly into two classes—the alkaline cyanide baths and the neutral or slightly acid baths. Arsenic is usually an essential

[1] These notes were first published in the Monthly Review of the American Electroplaters' Society, July, 1919.

[2] For a summary of such formulas, see O. P. Watts, Metal Industry, 12, p. 24; 1914.

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constituent of the alkaline baths; in fact, as has been shown and is confirmed by this work, the deposits from such baths may consist principally of arsenic, thus approaching in composition and properties the deposits obtained from "arsenic black" baths (prepared by the solution of arsenic trioxide in sodium hydroxide and sodium cyanide). Such deposits, therefore, hardly justify the term "black nickel," except as an indication that the solutions contain (or originally contained) nickel salts and are operated with nickel anodes. The neutral baths contain nickel sulphate or nickel ammonium sulphate, together with salts of zinc, copper, etc., and usually some sulphur compound, such as a sulphite, thiosulphate (hypo), or sulphocyanate. Of these, the sulphocyanate solutions have been most frequently used. The experiments and observations of the Bureau were confined chiefly to the alkaline cyanide baths and the sulphocyanate baths, which will be considered separately.

3. ALKALINE CYANIDE SOLUTIONS

It is probable that these baths were originally prepared (as they still are in some cases) by adding to a portion of the regular nickel-plating bath a solution of sodium arsenite—that is, arsenic trioxide dissolved in sodium hydroxide—forming a precipitate (essentially nickel arsenite) which was then dissolved by the addition of sufficient sodium cyanide. It is, therefore, not surprising that difficulty was experienced when efforts were made to state a definite formula for preparing such solutions from the original chemicals. In order to obtain clear solutions, or to produce the particular shade of deposit desired, other additions were made to the solutions, which resulted in very complicated and even irrational formulas. Perhaps the best illustration of such a formula is the so-called Rock Island formula (which, however, probably did not originate at the Rock Island Arsenal).

SOLUTION NO. 1

1. Dissolve 6 pounds of double nickel salts in 6 gallons of water.
2. Add 4 ounces of sulphuric acid.
3. Add 1 pound of "phosphoric zinc" (prepared by dissolving zinc in sirupy phosphoric acid diluted with an equal volume of water).
4. Add 5 pounds of carbonate of ammonia (a small amount at a time).
5. Add 4 pounds of sodium cyanide.

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SOLUTION NO. 2

1. Dissolve 10 pounds of arsenious acid and 10 pounds of caustic soda in 5 gallons of warm water.

Mix solutions No. 1 and No. 2 and dilute to 12 gallons.

Whatever the origin of this or equally complicated solutions suggested for military supplies, it is obvious that their use could not lead to satisfactory results. One of the difficulties experienced in attempting to follow some of the suggested formulas was that most of the metal in the salts was precipitated to the bottom, forming a sludge of cyanides, carbonates, or arsenites of nickel, zinc, etc. In one case after thorough mixing of the ingredients, only 7 per cent of the nickel added was found in the solution. Such solutions are, therefore, very wasteful, as well as unsatisfactory in operation.

In the effort to obtain a simple, satisfactory, alkaline "black nickel" solution, a large number of experiments were conducted, with frequent analyses of the solutions and the deposits. Assuming that the nickel is present as the double cyanide, and the arsenic as sodium arsenite, the simplest solution would consist of nickel sulphate treated with sufficient cyanide to dissolve the precipitate first formed, and arsenic trioxide with sufficient sodium hydroxide to dissolve it. Such solutions yielded, however, only unsatisfactory grayish deposits, which consisted principally of arsenic. When zinc sulphate was present in the solution more satisfactory deposits were obtained.

The exact function of the zinc is difficult to determine, since very little enters into the deposit. In the effort to employ certain of the commercial formulas, it was found that a very considerable excess of cyanide was necessary to obtain a clear solution. From such solutions, containing, for example, the ingredients here listed, good gray-black deposits were obtained.

Ingredients	Ounces per gallon	Grams per liter
Nickel ammonium sulphate (double salt)	6	45
Zinc sulphate	.5	3.8
Ammonium carbonate	5	37.5
Arsenic trioxide	.7	5
Sodium hydroxide	.7	5
Sodium cyanide	6	45

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On continued operation, however, the deposits became lighter in color, a large amount of sludge formed in the solution, and the latter changed very materially in composition, losing both arsenic and zinc. This is to be expected, since in these solutions nickel anodes are used and there is no opportunity for the deposited arsenic and zinc to be replaced. The nickel content decreases less rapidly, although nickel anodes are not readily corroded in cyanide solutions.

The net result of a large number of experiments and analyses upon alkaline "black nickel" baths was the conclusion that no bath of this type can produce uniformly satisfactory results unless it is controlled by very frequent and decidedly difficult chemical analyses, such as are not usually available in commercial plating plants. We do not feel justified, therefore, in recommending any formula for such solutions.

4. SULPHOCYANATE SOLUTIONS

A typical formula of this type, which has been used for many years and is given in numerous texts on plating, is as follows:

Ingredients	Ounces per gallon	Grams per liter
Nickel ammonium sulphate (double salt)	8	60
Zinc sulphate	1	7.5
Sodium [a] sulphocyanate	2	15

[a] Originally potassium or ammonium sulphocyanate was used commercially, but recently the sodium salt has almost entirely replaced it with equal satisfaction. (See C. H. Proctor, Metal Industry, 16, p. 26: 1918.)

Since it has been found in practice that such solutions give entirely satisfactory black deposits, the experiments conducted were directed to securing simple means of control of these solutions rather than to securing new or possibly better formulas.

Analyses of deposits produced from such solutions when freshly prepared showed a considerable variation in composition, due no doubt to slight differences in the original neutrality of the solutions and to slight fluctuations in the voltage.

In this, as well as in the alkaline "black-nickel" solutions, even slight changes in voltage produce marked differences in the proportion of the metals precipitated and hence in the color and physical properties of the deposits. Thus in the above sulphocyanate solution a good black deposit is produced with 0.5 to 0.7 volt (between anode and cathode); while if the voltage be in-

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creased to 1.5 to 2 volts, a light-colored, bright nickel deposit is secured. In the operation of such solutions, therefore, control of the voltage is more important than is the control of the current density. The latter probably will not, under good conditions, exceed 1 ampere per square foot (0.11 amp./dm²).

Good "black-nickel" deposits were found to contain from 40 to 60 per cent of nickel, 20 to 30 per cent of zinc, 10 to 14 per cent of sulphur, and 10 per cent or more of organic matter of undetermined composition. (The organic matter was formed by the the decomposition of the sulphocyanate upon electrolysis.) The ratio of sulphur to nickel was always less than that required for nickel sulphide, which contains about 65 per cent of nickel and 35 per cent of sulphur. It therefore appears reasonably certain that the black deposit consists essentially of a mixture of nickel sulphide, nickel, zinc, and organic matter.

If the zinc content of the solution is increased very much above that indicated by the above formula, grayish deposits, with high zinc content, are produced. If, on the other hand, the zinc content of the solution is greatly reduced or actually eliminated, the deposit is nearly pure nickel, with a light or bright nickel color. It is, therefore, important to maintain a fairly constant composition of solution, although good results can be obtained over a moderate range. In the normal operation of such a solution, the changes that are likely to occur are (*a*) a decrease in sulphocyanate, which must, therefore, be replenished at intervals; (*b*) a decrease in zinc content; and (*c*) an increase in nickel content. The latter arises from the fact that with nickel anodes, nickel is going into solution more rapidly than it is deposited, since part of the current is used in depositing zinc. At first it might appear possible to replenish the zinc and prevent an increase in nickel content by the substitution of zinc anodes for part of the nickel anodes. Attempts to do this, however, resulted in too high a zinc content, owing to the fact that zinc anodes are so much more readily corroded than nickel. The most practical way, therefore, to reduce the nickel content is to use nickel anodes which do not corrode very readily—for example, those with 96 per cent or more of nickel—and, if necessary, to restrict the anode surface.

In general, and especially when using high percentage nickel anodes, there is a tendency for the solution to become acid. Among the various additions which have been proposed to maintain neutrality are ammonia and nickel carbonate. Neither is

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entirely satisfactory, since addition of ammonia tends to precipitate the zinc from the solution unless great care is used, and, on the other hand, the use of nickel carbonate tends to still further increase the nickel content. For this purpose zinc carbonate should, therefore, be more satisfactory, since to the extent that it neutralizes the solution the zinc content will be maintained. Experiments have shown that by having an excess of zinc carbonate always present, good deposits may be secured over a considerable period. Whether in any case its use is sufficient to maintain the zinc content will depend upon the conditions of operation, and can be determined only by occasional analyses. Nickel, zinc, ammonia, and sulphocyanate may be determined in such solutions by the usual methods, which are not especially difficult where laboratory facilities and experience are available. Wherever possible, it is desirable to have such analyses made at intervals, although by observance of the above suggestions it is believed that the solutions will change in composition less rapidly than under the usual conditions.

5. APPLICATIONS OF "BLACK NICKEL"

In the application of "black nickel" plating to brass, a preliminary copper plating is frequently recommended or specified. It has not been found that deposits produced under such conditions are intrinsically more adherent or serviceable than those produced directly on brass. Where, however, the "Government bronze" effect is desired, the copper plating is an advantage in that whenever the black coating is removed—for example, by abrasion—an underlying copper coating produces a more pleasing effect than does brass. If the copper plating is so produced as to have a dull surface, it also tends to produce a matte effect in the "black nickel." This is frequently desired, especially when the sulphocyanate solution is used, which, on a polished surface, tends to produce a bright black finish. The actual wearing properties of the "black nickel" surface are largely determined by the quality of the lacquer subsequently applied.

Although "black nickel" may be deposited directly upon steel, it is more difficult to secure good results than if the steel is first plated with some other metal. For this preliminary coating, copper and nickel are most frequently used. Since, however, neither the copper nor the nickel, nor the "black nickel" deposits afford any appreciable protection against corrosion, finishes so

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produced are not desirable for outdoor exposure. For this purpose the steel should be first zinc plated, either in the sulphate or cyanide bath. Experiments have shown that steel plated with zinc and then with "black nickel" possesses about the same resistance to the salt spray test as does steel plated with the same thickness of zinc. In other words, the "black nickel" does not appreciably add to or detract from the protective value of the zinc coating.

6. CONCLUSION

A bath prepared from the ingredients here listed, kept neutral with an excess of suspended zinc carbonate, and operated with nickel anodes will give satisfactory black deposits upon brass, copper, and zinc; upon brass plated with copper; and upon steel plated with copper, nickel, or zinc. The deposit is probably a mixture of nickel, nickel sulphide, zinc, and organic matter of undetermined composition.

Ingredients	Ounces per gallon	Grams per liter
Nickel ammonium sulphate	8	60
Zinc sulphate	1	7.5
Sodium sulphocyanate	2	15

The authors desire to acknowledge the assistance of L. M. Ritchie and A. D. Bell in certain of the tests, and that of Dr. William Blum for suggestions and advice during the investigation.

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